SAN FRANCISCO CLEAN WATER PROGRAM
CITY AND COUNTY OF SAN FRANCISCO

BAYSIDE FACILITIES PLAN

NPDES PERMIT PROHIBITIONS ANALYSIS REPORT

MARCH 1980

cgkt CALDWELL-GONZALEZ-KENNEDY-TUDOR
CONSULTING ENGINEERS

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March 31, 1980

Mr. Harold C. Coffee, Jr.
Project Manager, Bayside Facilities
San Francisco Clean Water Program
150 Hayes Street, Sixth Floor
San Francisco, California 94102

500-50/19

Subject: Bayside Facilities Plan NPDES Permit Prohibitions Analysis Report

Dear Mr. Coffee:

The California Regional Water Quality Control Board, San Francisco Bay Region, has placed a prohibition against obtaining less than 10:1 initial dilution of combined sewer overflows and against discharge of combined sewer overflows into confined receiving bodies of water (dead-end sloughs) until such time that it can be shown that the costs of achieving these requirements are inordinate. This report addresses these issues and presents the results of the field study program. A cost-effectiveness analysis of the control of combined sewer overflows is presented and recommendations are made concerning the facilities in each drainage basin.

Very truly yours,

Robert L. Mills

Robert L. Mills
Assistant General Manager

RLM:ch

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CHAPTER 1

INTRODUCTION

The City and County of San Francisco has 39 points of combined sewer overflow around its periphery. All but one of these overflows discharge directly at the shoreline, and a few discharge into confined bodies of water (sloughs or channels) of San Francisco Bay. These discharges are sporadic and occur only about 3 percent of the total time.

The Environmental Protection Agency has issued Program Requirements Memorandum RPM No. 75-34, which states that combined sewer overflow projects will be funded only when careful planning has demonstrated that they are cost-effective. It must be shown that the pollution control technique proposed for combined sewer overflow is a more cost-effective means of protecting the beneficial use of the receiving waters than other combined sewer pollution control techniques, and that "the marginal costs are not substantial compared to marginal benefits."

The California Regional Water Quality Control Board, San Francisco Bay Region, has placed a prohibition against obtaining less than 10:1 initial dilution of combined sewer overflows and against discharge of combined sewer overflows into confined receiving bodies of water (dead-end sloughs) until such time that it can be shown that the costs of achieving these requirements are inordinate. These prohibitions are based solely on aesthetic effects and not on physical, chemical, or bacteriological quality of receiving waters or sediments. The Water Quality Control Plan, San Francisco Bay Basin, State Water Resources Control Board, states for wet weather overflows that:

> Water quality objectives require that all outfalls achieve an initial dilution of 10:1 in order to minimize adverse aesthetic effects of discharge, especially that of untreated or partially treated overflows. It is recommended that any possible wet weather overflow, whether from a separate or combined system, should receive coarse screening to remove large visible floating material and to protect the outfall system than be discharged through outfalls which satisfy the 10:1 dilution objective. Overflow locations should be in areas where discharge will cause minimal effects on beneficial uses. Removal of such overflow locations from dead-end sloughs and channels, and close proximity to marinas and land beaches is especially desirable. In no case shall

untreated or partially treated wet weather discharges be tolerable where local currents or confinement will result in accumulation of floatable materials.

This report addresses these issues and presents a cost-effectiveness analysis of the control of combined sewer overflows. The results presented in this report are preliminary and will be refined or revised, as necessary, after a more thorough analysis can be performed during the continuing Bayside Facilities Planning Project.

The analysis presented in this report is based on a "worst case" condition which assumes that no storage is available in the collection and transport system. A more realistic evaluation will be presented as soon as a detailed hydraulic analysis of the system can be performed.

CHAPTER 2

NPDES PERMIT REQUIREMENTS

The current requirements and conditions for wastewater discharges from wet weather diversion structures of the City and County of San Francisco are contained in National Pollutant Discharge Elimination System (NPDES) Permit No. CA0038415 for the Richmond-Sunset Sewerage Zone and in NPDES Permit No. CA0038610 for the North Point and Southeast Sewerage Zones. Included in both of these NPDES Permits are the following prohibitions:

> A.2 - Discharge of waste into dead-end sloughs or similar confined water areas or their tributaries is prohibited. A.3 - Discharge of waste at any point where it does not receive a minimum initial dilution of at least 10:1 is prohibited.

The permits further state:

> Exceptions to prohibitions 2 and 3 will be considered where an inordinate financial burden would be placed on the discharger relative to beneficial uses protected and when an equivalent level of environmental protection can be achieved by alternate means.

Also, it is further stated that:

> Further mitigation may be required in the future, after facilities are placed in operation, if it is determined that beneficial uses are not adequately protected.

California Regional Water Quality Control Board, San Francisco Bay Region, Order No. 79-119, requires that the City and County of San Francisco submit a report by March 1, 1980, on facilities needed for compliance with prohibitions A.2 and A.3, or demonstrate that an exception is warranted.

CHAPTER 3

FIELD STUDY PROGRAM RESULTS

The field study program to provide data on the impact of overflows on beneficial use areas includes the following three elements: dye tracer survey, float tracking survey, and water sampling for coliform analyses. The purpose of the dye tracer survey program was to obtain estimates of initial dilution achieved by shoreline overflow discharges into relatively deep water in the bay and across the beach into the surf zone in the ocean. The overflow structure at Howard Street represents the first type of these discharges in that it terminates at the seawall boundary along the bay front in relatively deep water. Howard Street was chosen because the zone of initial dilution around the end of the outfall is accessible both from the street and by boat. The overflow at Lincoln Way represents the oceanside overflow outfall. Scope of work of the dye survey was to involve injecting dry tracer in measured amounts into the overflow upstream of the outlet and measuring resultant diluted dye concentrations in the receiving waters at the end of the overflow jet. Plans also included taking aerial photographs of the dye plume following discharge. Initial dilution is calculated from the ratio of dye concentration in the waste stream before discharge to measured dye concentration at the end of the overflow jet.

The purpose of the float tracking survey was to estimate where floatable materials associated with overflows might wash ashore in specified beneficial use areas and to measure the time interval of travel from the outfalls to the shore. Scope of work was to involve releasing color-coded wooden floats into the receiving waters at bayside and oceanside overflow outfall locations and subsequently walking the shoreline areas over a 4-day period, noting both the color and location of floats found. The data, indicating areas of heaviest or more frequently occurring floatable impingement, were to be used in selecting coliform sampling stations.

The coliform sampling program was designed to analyze waters of beneficial use areas along city beaches for total and fecal coliform organisms. Sampling took place during and following overflows to determine expected peak concentrations and the time required for coliform concentrations to decrease to background levels following a storm. Scope of work was to begin collecting water samples at slack water before flood tide within 12 hours of start of overflow and to continue sampling approximately at 12-hour intervals until 3 days following end of overflows.

DYE TRACER SURVEYS

A dye tracer survey to measure initial dilution of the Howard Street overflow discharge into the bay was conducted on January 11, 1980, by injecting liquid dye through a manhole into the overflow stream and measuring the resultant concentration in the bay at the end of the overflow jet.

The Howard Street overflow (diversion) structure is a 7-foot diameter pipe terminating at the seawall at the north side of Pier 16. Pipe crown elevation is 6.75 feet above mean lower low water (MLLW). Higher high water tide elevations in the bay may typically reach 7 feet above MLLW; hence, the Howard Street overflow is essentially a surface discharge.

Predicted peak (short duration) flow from 5-year storm during which rainfall intensity is 1.5 inches per hour (in./hr), is 175 million gallons per day (mgd) (California Regional Water Quality Control Board National Pollutant Discharge Elimination System (NPDES) Permit CA0038610). Flow typical of a 1-year storm is approximately 60 mgd (City of San Francisco, personal communication). Rainfall preceding the January 11 dye survey had been light and was stopping at about the time dye injection began. The sewer, however, continued to overflow until after dye injection was completed. Calculated average amount of overflow during the 30-minute-long interval of dye injection was approximately 8 mgd. This calculation is explained later.

Tide conditions at the time of the survey were as follows: times of high and low tide were, respectively, 1010 and 1710 hours; times of slack water before ebb, peak ebb, and slack water before flood were, respectively, 0730, 1040, and 1440 hours.

Dye injection involved continuing pumping at 18.9 liters of E. I. Dupont Rhodamine WT dye liquid at a rate of approximately 630 milliliters per minute (ml/min) between 0945 and 1015 hours directly into the overflow stream through a manhole in the center of Howard Street, west of the intersection with the Embarcadero. Grab samples for analysis to determine the overflow rate and initial dye concentration in the overflow stream were taken from a second manhole located in the roadway of the Embarcadero, approximately 10 feet from the terminus of the overflow structure. Samples were also taken from the waste stream immediately after it entered the bay and before it had a chance to mix with bay waters. A total of six grab samples were taken between 1007 and 1015 hours. Dye concentrations, measured with a Turner Designs Model 10-005 fluorometer, ranged from 3,880 parts per billion (ppb) to 8,660 ppb. Average concentration was 6,087 ppb. Undiluted dye concentration is 200,000 parts per million. The product of dye injection rate times undiluted dye concentration, divided by the average measured dye concentrations in the effluent (6,087 ppb), yields the average effluent flow rate.

As the dye field spread out into the bay, it moved northward alongshore with the ebbing tidal current. Figure 3-1 shows the field at successive times during the survey. A survey vessel with the Turner fluorometer on board made repeated traverses of the dye field. Water samples were withdrawn continuously from a depth of 0.7 to 1 meter below the surface and pumped through the fluorometer whose output was recorded on a strip chart recorder. The sampling depth was chosen as representative of the surface layer. An earlier overflow dye study (CH2M HILL, June 1979) indicates the surface layer may be 1 to 2 meters deep.

From the survey vessel, the observers could detect the overflow jet in the bay. Positioning the vessel at the apparent end of this jet, approximately 75 meters from the shoreline, they recorded dye concentration at various times throughout the survey in order to obtain dye concentration measurements with which to compute overflow initial dilution. These peak concentrations observed were at 1010, 1017, and 1025 hours. Concentrations, respectively, were 370, 350, and 390 ppb. The average of these is 370 ppb. The average initial dilution, therefore, is 6087/370 ppb or 16:1.

Aerial photographs were not taken due to below-minimum flying conditions.

<u>Shoreline Discharge Initial Dilution</u>

In 1978, the State Water Resources Control Board (SWRCB) published Table B guidelines to the California Water Quality Control Plan which presented a method for estimating the minimum initial dilution resulting from surface discharge of a buoyant effluent. The method is based on a mathematical model developed at the Massachusetts Institute of Technology for three-dimensional heated surface discharge computations. The model assumes that the receiving water body is large such that neither the side nor bottom boundaries interfere with development of the surface jet. Initial dilution, according to the SWRCB definition, is completed when turbulent entrainment due to momentum ceases. This point occurs at the end of the region of stability in which centerline jet velocity drops sharply, jet depth decreases, jet lateral spread increases, and wastewater concentration remains relatively constant.

In applying the SWRCB method to the Howard Street overflow discharge, additional assumptions and input parameters are as follows:

1. The pipe terminus remains covered and the pipe crown is at surface water level.

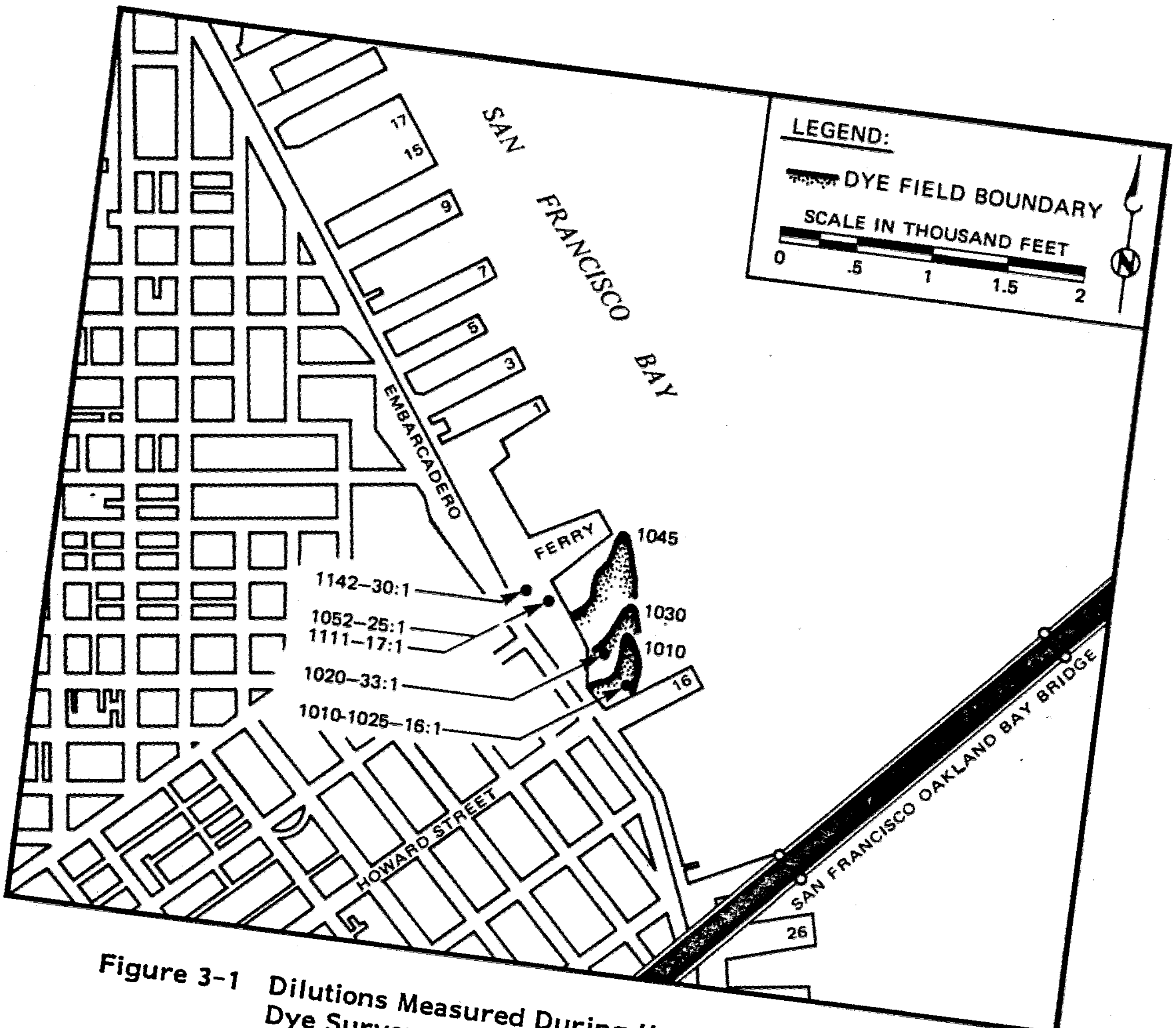

Figure 3-1 Dilutions Measured During Howard Street Overflow Dye Survey, January 1980

2. The pipe end is open and not covered by a flap gate.

3. Measured wastewater temperature and salinity are 12 degrees Centigrade (C) and 1 parts per thousand (ppt), respectively (CH2M HILL, June 1979).

4. Measured receiving water temperature and salinity are 12 degrees C and 25 ppt, respectively.

5. Wastewater discharge velocity ranges from 0.7 to 7.0 feet per second (fps). Corresponding wastewater flow ranges from 1.75 to 175 mgd.

The results are that minimum initial dilution ratio ranges from less than 1 (little mixing with receiving water) for low waste flows to approximately 5 (5 to 1) for a waste flow of 7 feet per second.

Observed initial dilution was higher than what the SWRCB method would predict. A flap gate covers the pipe end in order to restrict tidal intrusion into the sewer in the absence of overflows. If only partially opened, this could cause increased flow speeds which account for higher dilutions.

The difference between 16 to 1 and 5 to 1 initial dilution is not significant. Although both the SWRCB mathematical method and field survey obtain estimates for initial dilution, the values are equivalent to one another. Whereas the mathematical method predicts dilution at fixed points in space, the end of the zone of initial dilution, in reality, is not a point. This zone, instead, is a narrow area separating the region where jet momentum is visually apparant from the larger region where jet momentum is absent and ambient currents in the receiving waters are predominant in causing further mixing.

Shoreline Versus End of Pier Discharges

The difference between shoreline and end-of-pier discharges is seen in comparing Figure 3-1, showing the Howard Street dye survey, and Figure 3-2, showing the results of a dye survey conducted at the North Point Water Pollution Control Plant (NPWPCP) in 1970. The 1970 survey involved a continuous, 10-hour dye injection into the wastestream at the plant. The effluent is discharged through four 48-inch outfall lines, two of which are suspended under Pier 33 and two under Pier 35. At that time, the lines terminated in 45-degree downward elbows about 10 feet below the surface at the end of the pier. They now have diffusers. Minimum initial dilutions calculated from surface grab samples collected over the effluent boil were about 3:1 or 4:1. Dilutions were 20:1 within a distance of 50 feet from the boil, and were not less than

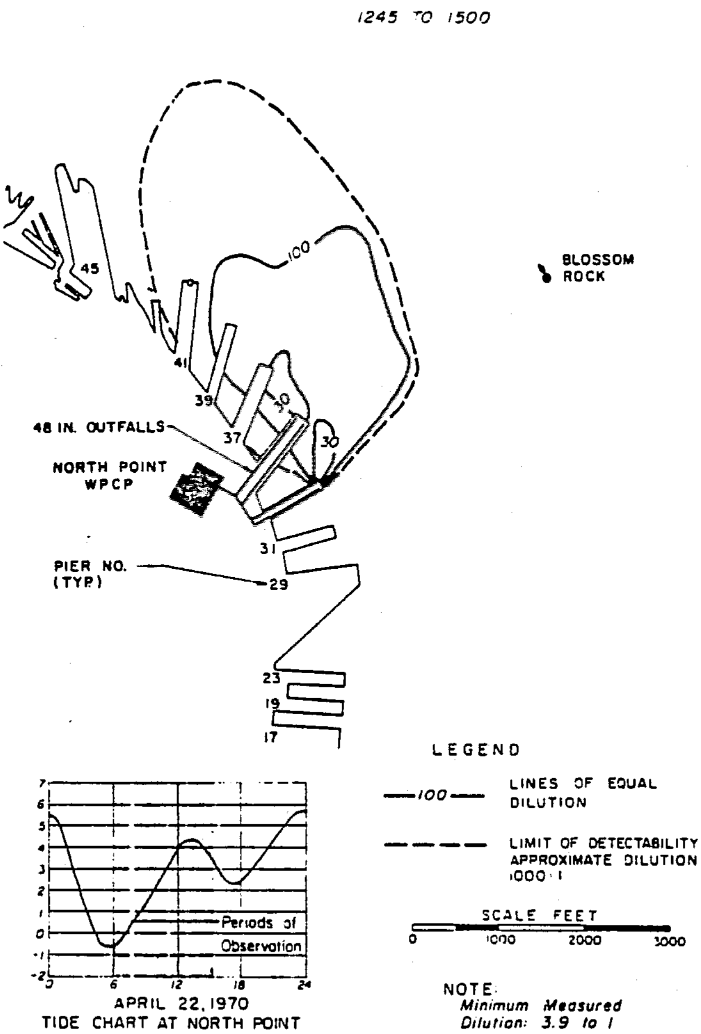

Figure 3-2 Dilutions Measured During North Point Treatment Plant Outfall Dye Study, April 1970

30:1 beyond 600 feet from the boil. As Figure 3-2 shows, dilutions in the wastewater field along its contact with the shoreline were 100:1 or greater.

In contrast, the Howard Street shoreline discharge showed that the wastewater field remained very close to shore and barely extended beyond the pierhead line (imaginary line connecting pier ends). Dilutions along the shore were 17 to 1 to 30 to 1.

The Lincoln Way dye survey has not been conducted to date and is pending occurrence in an overflow and optimum tide conditions during daylight hours.

FLOAT TRACKING SURVEY

A field survey, involving tracking of wooden floats (thin wooden sticks, used as coffee stirrers, painted with fluorescent colors) released at various overflow discharge locations, provided information on impingement of overflow-derived floatables in shoreline recreational areas. Figure 3-3 shows overflow locations and shoreline areas included in the study. Color coding of floats indicated the overflow release location and the time of release at slack before ebb and at slack water before flood. Table 3-1 shows tidal stage times throughout the survey period. Releases were between 0635 and 0725 hours, and between 1330 and 1400 hours, on January 18. Each release consisted of approximately 1,000 or more wooden sticks, dropped from a helicopter which hovered above the overflow discharge areas. The number and location of floats found during the 4 days following their release was recorded. These data indicate the relative impact on a particular area by nearby overflows, and the interval between time of discharge and time of impingement. With one exception, as explained in Table 3-2 footnote, all floats were picked up as they were found. Results of the float survey are given in Table 3-2 and on Figure 3-4. In two instances, the number of floats recovered exceeded 1,000. The number of sticks in each release is only approximate. The sticks were purchased in boxes of at least 1,000 each. It is very likely that each box may have contained slightly more than 1,000. We did not count each box and assumed that each box contained a number sufficiently close to 1,000.

The largest percent return of floats was on the ocean side. Essentially, 100 percent return was observed from the Vicente Street, Lincoln Way, Lobos Creek (Bakers Beach), and Baker Street releases. For the most part, these floats washed ashore within a short distance of their point of release. At Vicente Street, Lincoln Way, and Lobos Creek Outfalls, releases were near the outer edge of the surf zone, thus wave action was responsible for bringing the floats ashore. Maximum excursions from these

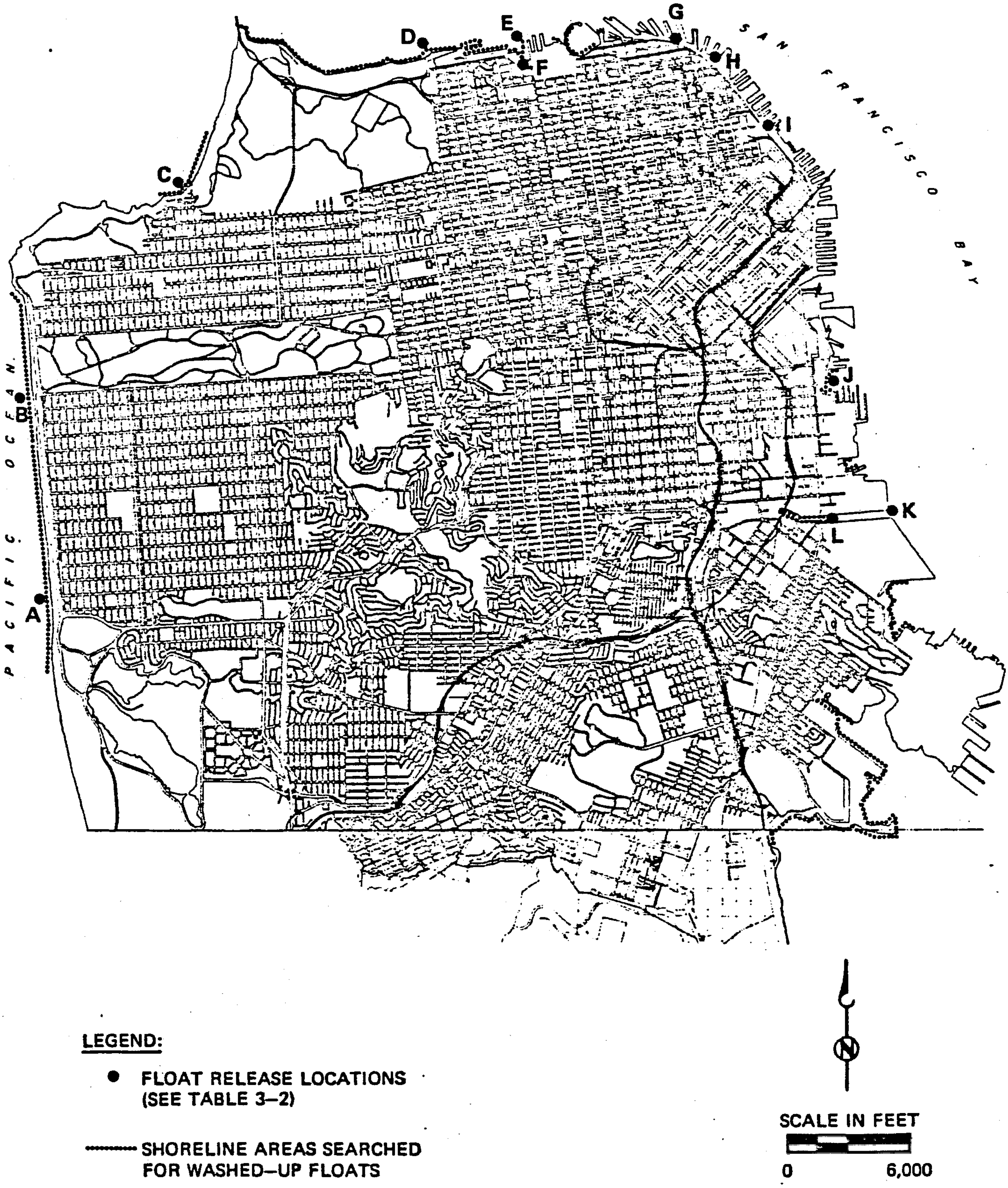

Fig. 3-3 Float Survey Study Area

Table 3-1. Tidal Stage Times at Golden Gate During Float Survey

Day	SBE[a]	Peak ebb	SBF[b]	Peak flood
January 18	0200 1305	0415 1625	0720 2020	1015 2320
January 19	0245 1355	0500 1715	0812 2102	1105
January 20	 0330 1450	 0555 1805	 0905 2150	0005 1155
January 21	 0415 1550	 0640 1855	 1010 2340	0050 1255

[a]SBE = Slack before ebb.
[b]SBF = Slack before flood.

Note: Compared to Golden Gate, tidal stage occurs earlier by 75 to 80 minutes off Ocean Beach, by 20 to 25 minutes south of Alcatraz, by 10 to 30 minutes off Rincon Point, and by 20 to 40 minutes off Point Avisadero.

Table 3-2. Float Survey Data

Washed-up float location	Date of recovery	Observation time interval		Origin of float[a]	Number of floats found
		From	To		
Vicente Street vicinity	1/18/80	0600	1200	A	105[b]
		1200	1800	A	984
	1/19/80	0600	1200	A	39
				B	1
Lincoln Way vicinity	1/18/80	0600	1200	B	664[b]
	1/19/80	0600	1200	B	839
	1/21/80	1200	1800	A	1
				B	61
Phelan Beach and Baker Beach	1/18/80	0600	1200	C	1,896[c]
				H	4
				D	1
				G	5
	1/21/80	1200	1800	I	1
				E	1
				C	167
				H	2
				D	1
				G	5
				F	2
Fort Point to east edge of Crissy Field	1/18/80	1200	1800	H	25
	1/19/80	0600	1200	D	19
				H	12
		1200	1800	E	2
				D	42
				H	25
				G	1
	1/20/80	1200	1800	E	13
				F	3
				D	16
				H	14
				I	6
				G	15
	1/21/80	1200	1800	E	17
				F	8
				D	24
				H	16
				I	19
				G	21
East edge of Crissy Field to Gashouse Cove	1/18/80	1200	1800	E	384
				F	80
				D	908
				H	23
	1/19/80	0600	1200	E	377
				F	25
				D	4
				H	4

[a]Key:

A - Vicente Street
B - Lincoln Way
C - Lobos Creek
D - Baker Street
E - Mouth of Gashouse Cove
F - Laguna Street - at discharge point
G - Beach Street
H - Sansome Street
I - Jackson Street
J - Mariposa Street
K - Islais Creek
L - Third Street Bridge

[b]The number of floats of one color code found exceeds 1,000 because the release included more than 1,000. See text.

[c]Floats at Phelan Beach and Baker Beach were not picked up when first found and instead were left on the beach. As a result, floats initially counted after 0600 were counted again before 1200, thus giving a misleading account of total floats found.

Table 3-2. Float Survey Data (continued)

Washed-up float location	Date of recovery	Observation time interval From	To	Origin of float[a]	Number of floats found
East edge of Crissy Field to Gashouse Cove (continued)	1/20/80	0600	1200	E	57
				F	23
				D	7
				H	7
				I	6
				G	20
	1/21/80	1200	1800	E	20
				F	12
				D	6
				H	4
				I	7
				G	7
Aquatic Park	1/18/80	1200	1800	H	16
	1/19/80	0600	1200	G	4
				H	16
				I	2
	1/20/80	0600	1200	G	13
				H	18
				I	12
	1/21/80	0600	1200	E	2
				F	10
				G	22
				H	11
				I	6
Mariposa Street	1/18/80	0600	1200	-	0
		1200	1800	-	0
	1/19/80	1200	1800	J	75
Warm Water Cove	1/18/80	0600	1200	-	0
		1200	1800	-	0
	1/19/80	1200	1800	K	58
				L	10
				J	1
Islais Creek	1/18/80	0600	1200	-	0
		1200	1800	L	3
	1/19/80	1200	1800	L	42
India Basin	1/18/80	0600	1200	K	95
	1/19/80	1200	1800	K	8
Candlestick Park	1/19/80	0600	1200	-	0

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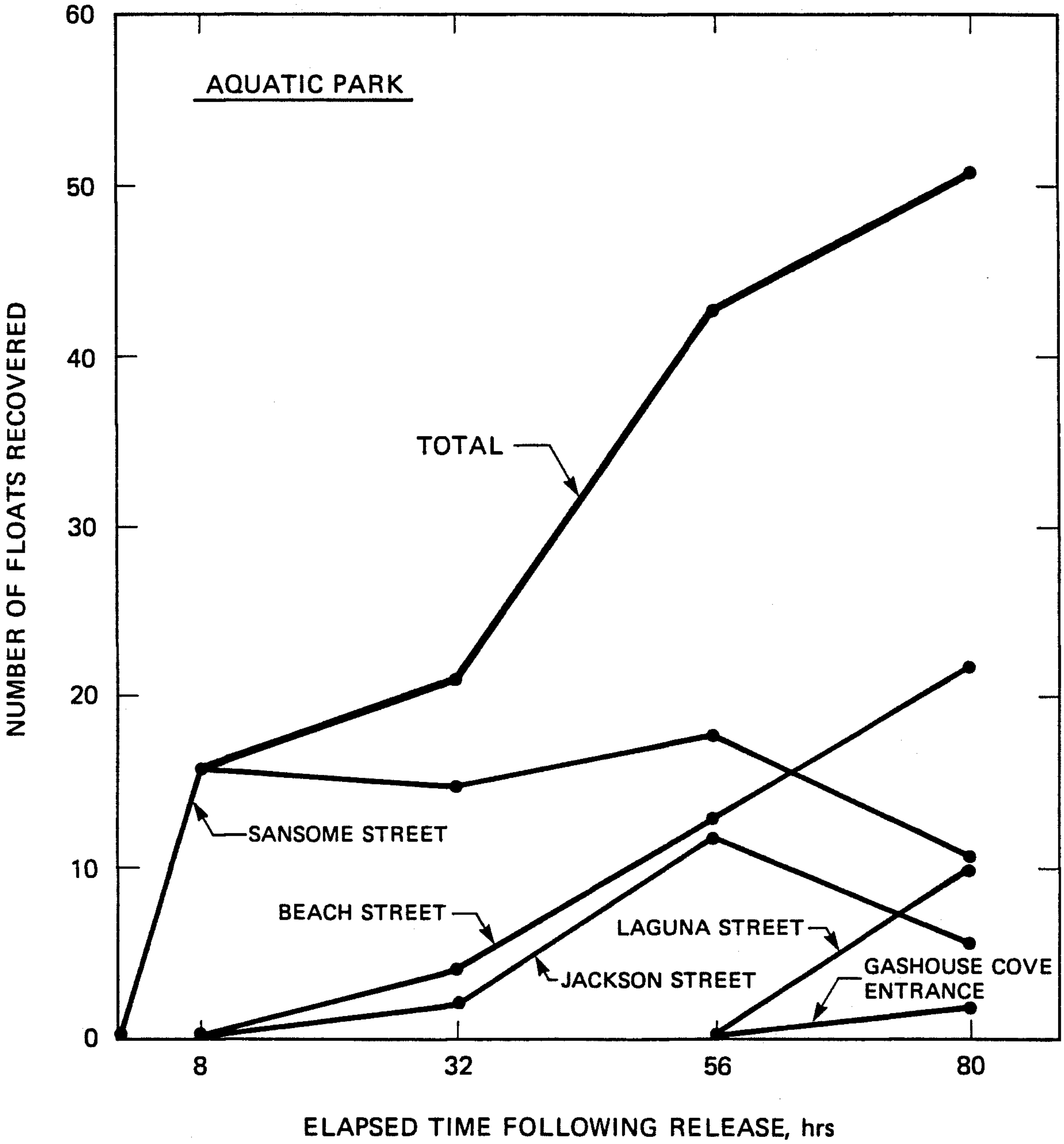


Figure 3-4 Floats Found at Aquatic Park

outfalls were 700 meters south and 600 meters north at Vicente Street, 200 meters south and 300 meters north at Lincoln Way, and approximately 200 to 300 meters either side of Lobos Creek.

Floats found along the shore from Fort Point to the east edge of Crissy Field originated from Baker Street (15 percent return), Sansome Street (4.5 percent return), and Beach Street, mouth of Gashouse Cove, Jackson Street, and Laguna Street (each less than 2 percent return).

Floats found along the shoreline from the east edge of Crissy Field to Gashouse Cove originated from Baker Street (45 percent return), mouth of Gashouse Cove (42 percent return), Laguna Street (7 percent return), and Sansome Street, Jackson Street, and Beach Street (each less than 2 percent return).

At Aquatic Park, floats found washed ashore originated from Beach Street and Sansome Street (each 2 percent return), and Laguna Street and Jackson Street (less than 0.5 percent). Figure 3-4 shows the relative time of arrival of floats at Aquatic Park and indicates that overflow proximity is not the sole factor determining when coliform organisms might arrive at Aquatic Park following an overflow. Beach Street is closer than Sansome Street to Aquatic Park, but circulation in the vicinity of the overflow is probably less due to presence of the Marina breakwaters. Gashouse Cove is also closer, but movement back into the bay during flood tide is apparently less than ebb-tide movement.

Along the southeast bay side, floats from Islais Creek washed up at Warmwater Cove and India Basin, but were not found in Islais Creek itself. No floats were found at Candlestick Park.

COLIFORM ANALYSES

Results of the float survey provided information used in planning the coliform sampling program. Shoreline locations where floats consistently washed ashore suggested preferred coliform sampling stations. The long interval found between time of discharge and time of float arrival at certain beach areas suggested that the sampling should be carried out for several days following the end of overflows. Figure 3-5 shows overflow locations and Figure 3-6 shows coliform sampling stations.

Table 3-3 gives the sampling schedule followed during the survey. Sampling was timed to occur near time of slack water before flood, except on February 15 and 17, when additional samples were taken at slack before ebb to provide information on tidal effects. Choice of slack-before-flood sampling resulted from the

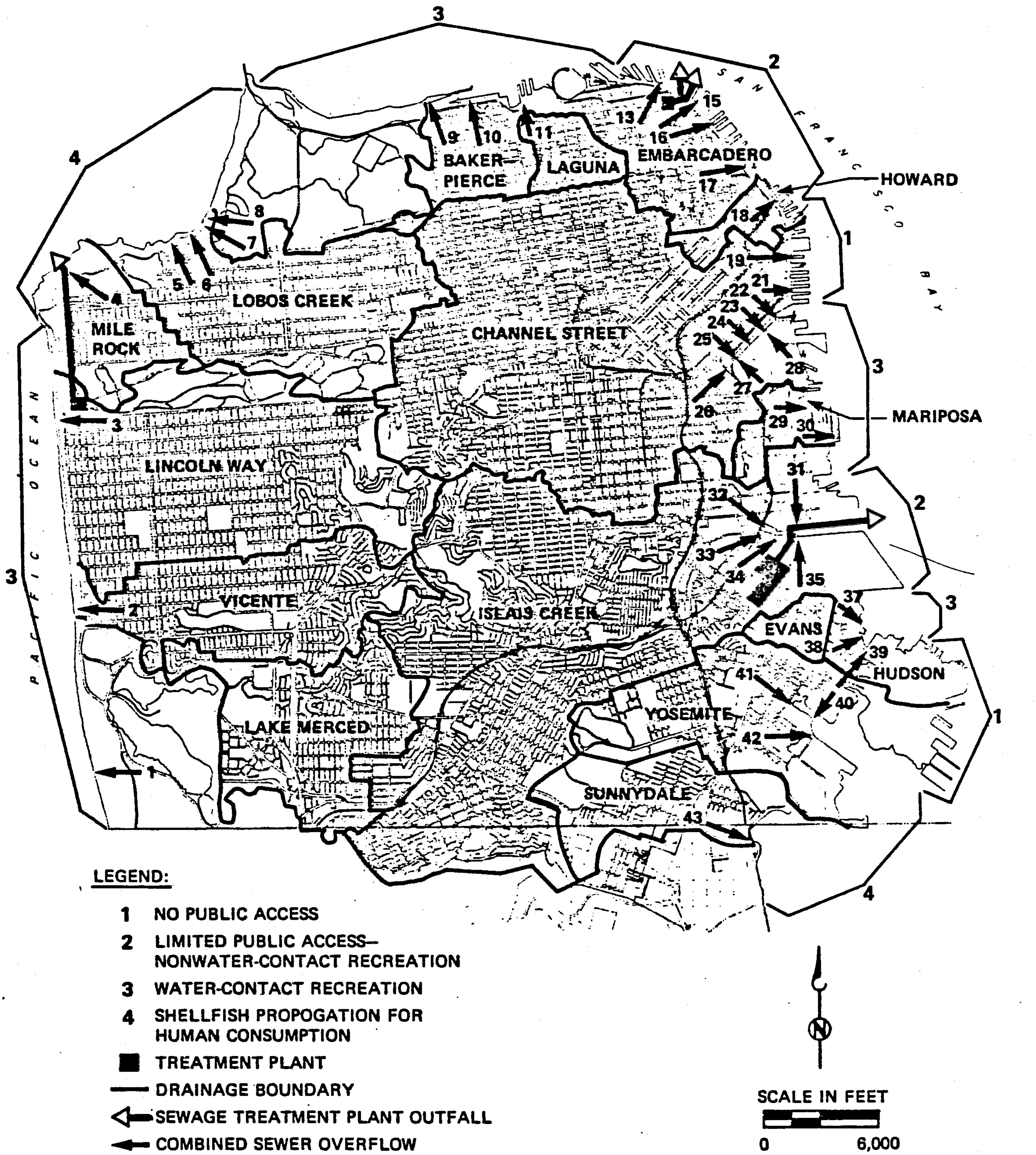


Fig. 3-5 Sewer Overflow Locations

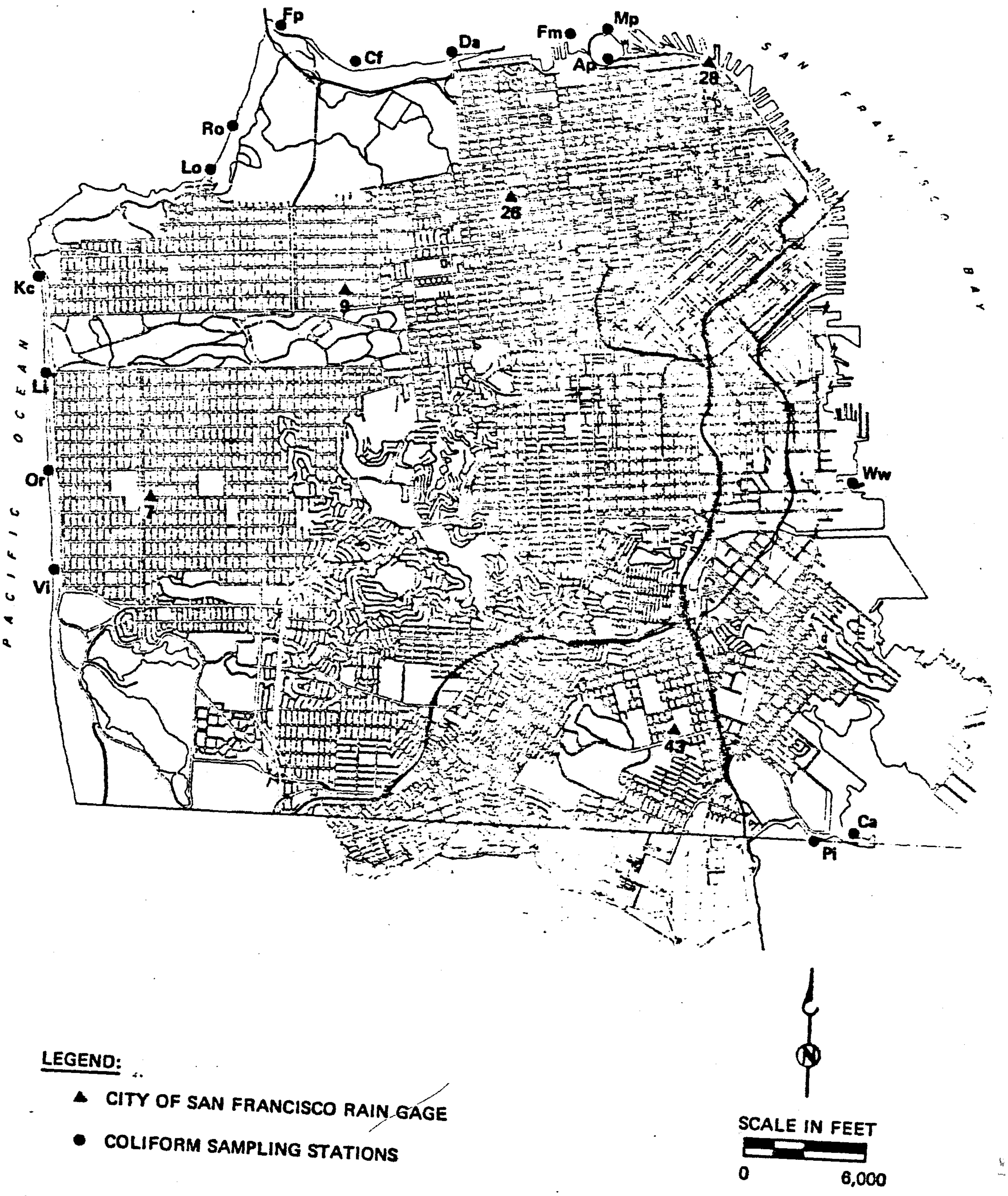


Fig. 3-6 Coliform Sampling Stations and Rain Gage Locations

Table 3-3. Scheduled Times of Coliform Sampling, February 15-24, 1980

Date	Time interval		Tidal stage	
	From	To	SBE	SBF
February 15, 1980	1200	1300	X	
	1800	1900		X
February 16, 1980	0600	0700		X
	1900	2000		X
February 17, 1980	0200	0300	X	
	0700	0800		X
	1230	1330	X	
	1930	2030		X
February 18, 1980	0730	0830		X
	2000	2100		X
February 19, 1980	0830	0930		X
	0945	1145		X
February 21, 1980	1100	1300		X
February 22, 1980	1215	1415		X
February 23, 1980	1315	1515		X
February 24, 1980	1415	1615		X

Note: SBE = slack before ebb
SBF = slack before flood

float survey data which showed that most floats found at Aquatic Park and Crissy Field, two important recreational areas, originated from overflow locations farther inside the bay.

Table 3-4 presents bacteriological analysis results for total and fecal coliform organisms. Highest count was 160 million MPN/ 100 ml at Lincoln Way. Other high counts, greater than 100,000 MPN/100 ml, in decreasing ranking order were at Lobos Creek, Candlestick fishing pier, Vicente Street, and Kellys Cove. Of these, only Lincoln Way, Lobos Creek, and Vicente Street are outfall locations. High total with low fecal counts probably means lesser influence by an overflow than to widespread background conditions in the bay following a storm. Nonpoint source surface runoff into the bay is likely a major contributing source for these conditions. High total counts may also be related to high Delta outflow which, because of the heavy rain, was increased due to upland runoff. This is particularly the case for the North Bayside area out past Golden Gate.

Data in Table 3-4 are summarized in Table 3-5, which presents number of days, out of a possible total number of 10 days, coliform levels exceeded 10,000 and 1,000 MPN/100 ml. Total coliform levels exceeded 10,000 most frequently at Lobos Creek and Candlestick fishing pier, which is near Sunnydale overflow. The number of days that counts exceeded 1,000 was about equal for total and fecal coliform levels along the ocean side and grossly unequal in the bay. This difference emphasizes the contribution of general background conditions rather than specific overflows. Lobos Creek exceeded 10,000 fecal coliforms more often than any of the other sampling locations, which is probably due to the higher frequency of overflows there than at other locations. Candlestick fishing pier frequently exceeded 10,000 and 1,000 because of its close proximity to the Sunnydale overflow. In the Sunnydale area, the route of water mass movement past the outfall and the fishing pier is apparently direct.

Figures 3-7 through 3-11 present graphs of total coliform concentrations as a function of time during the 10-day sampling survey. The graphs are grouped by area. Figures 3-7 and 3-8 show that at Ocean Beach and Bakers Beach, concentrations decreased sharply after overflows ceased as a result of the intense flushing caused by tidal flow and wave-induced longshore currents sweeping the area. Concentrations are less variable on the bay side, except at the Candlestick fishing pier which is located near the Sunnydale overflow.

Superimposed on Figures 3-7 and 3-11 is the rainfall data. The data are from one city raingage located within the drainage area of the overflows. Hourly rainfall amounts were aggregated over 4-hour intervals for purposes of graphical display. Rainfall started on February 14, 1980, and ended on February 27, 1980. The rainfall resulted from six major storms, separated by at least 8 hours.

Table 3-4. Coliform Survey Results, MPN/100 ml

Sampling stations		Date: 2/15/80 Time: 1225-1500		Date: 2/15/80 Time: 1730-1850		Date: 2/16/80 Time: 0600-0800		Date: 2/16/80 Time: 1900-2030	
Location	Code	Total	Fecal	Total	Fecal	Total	Fecal	Total	Fecal
Vicente Street	Vi	330,000	8,000	5,000	<2,000	17,000	14,000	11,000	11,000
Ortega Street	Or	13,000	1,700	2,300	500	<200	<200	17,000	1,700
Lincoln Way	Li	280,000	14,000	7,000	<2,000	160,000,000	1,400,000	310,000	13,000
Kellys Cove	Kc	110,000	3,300	1,400	<200	3,300	200	33,000	3,300
Lobos Creek	Lo	1,300,000	230,000	43,000	5,000	940,000	70,000	33,000	5,000
Bakers Beach	Ro	23,000	4,900	2,300	200	2,300	200	3,300	490
Fort Point	Fp	2,200	110	2,300	230	790	70	3,300	490
Crissy Field	Cf	7,900	<200	3,300	800	2,300	200	1,300	170
Baker Street	Da	2,300	790	14,000	1,700	4,600	<200	4,900	1,100
Fort Mason	Fm	1,300	<200	33,000	2,300	1,200	200	2,300	490
Municipal Pier	Mp	13,000	2,300	2,300	500	1,700	<200	330	130
Aquatic Park	Ap	2,100	500	7,000	1,300	3,300	200	11,000	1,700
Warmwater Cove	Ww	4,600	400	4,900	400	11,000	<200	4,600	1,300
Candlestick Park	Ca	17,000	<2,000	13,000	8,000	17,000	<2,000	11,000	1,300
Candlestick Fishing Pier	Pi	130,000	23,000	230,000	13,000	23,000	2,000	490,000	70,000

Table 3-4. Coliform Survey Results, MPN/100 ml (continued)

Sampling stations		Date: 2/17/80 Time: 0135-0410		Date: 2/17/80 Time: 0650-0805		Date: 2/17/80 Time: 1215-1400		Date: 2/17/80 Time: 1915-2115	
Location	Code	Total	Fecal	Total	Fecal	Total	Fecal	Total	Fecal
Vicente Street	Vi	500	<200	500	<200	200	<200	13,000	3,300
Ortega Street	Or	490	<20	330	20	330	50	2,300	790
Lincoln Way	Li	<2,000	<2,000	<2,000	<2,000	130,000	4,900	4,900	800
Kellys Cove	Kc	1,100	<200	200	<200	3,100	700	14,000	2,300
Lobos Creek	Lo	<2,000	<2,000	<2,000	<2,000	230,000	49,000	4,900	500
Bakers Beach	Ro	3,300	490	790	130	4,900	2,300	3,300	330
Fort Point	Fp	790	130	790	220	330	50	2,300	1,300
Crissy Field	Cf	490	70	330	50	13,000	80	1,700	140
Baker Street	Da	2,200	40	790	110	3,100	3,100	3,100	1,100
Fort Mason	Fm	1,300	160	490	330	1,100	330	1,400	210
Municipal Pier	Mp	790	490	1,100	70	330	330	790	330
Aquatic Park	Ap	2,300	60	1,300	330	790	490	790	330
Warmwater Cove	Ww	3,300	790	17,000	490	3,300	310	4,600	790
Candlestick Park	Ca	11,000	310	23,000	1,700	11,000	1,700	17,000	2,300
Candlestick Fishing Pier	Pi	4,900	700	70,000	13,000	70,000	1,300	33,000	13,000

Table 3-4. Coliform Survey Results, MPN/100 ml (continued)

Sampling stations		Date: 2/18/80 Time: 0715-0905		Date: 2/18/80 Time: 1945-2125		Date: 2/19/80 Time: 0810-1020		Date: 2/20/80 Time: 0855-1200	
Location	Code	Total	Fecal	Total	Fecal	Total	Fecal	Total	Fecal
Vicente Street	Vi	4,900	2,200	110	20	130,000	49,000	330	130
Ortega Street	Or	230	20	110	<20	3,300	700	790	40
Lincoln Way	Li	800	<200	200	<20	1,800,000	700,000	1,100	49
Kellys Cove	Kc	4,900	<200	330	230	11,000	4,900	1,700	80
Lobos Creek	Lo	2,200	<200	3,500,000	1,100,000	330,000	49,000	4,900	330
Bakers Beach	Ro	3,300	<20	1,700	230	49,000	23,000	4,900	230
Fort Point	Fp	1,700	20	4,900	1,100	17,000	790	460	80
Crissy Field	Cf	490	40	3,300	490	3,300	490	1,300	230
Baker Street	Da	7,900	50	17,000	1,300	23,000	2,200	7,900	2,300
Fort Mason	Fm	490	<20	3,300	700	7,900	1,300	790	110
Municipal Pier	Mp	700	20	1,400	630	3,300	1,300	1,100	50
Aquatic Park	Ap	2,300	20	1,100	330	4,900	490	460	230
Warmwater Cove	Ww	7,000	80	7,900	790	3,300	2,300	4,900	700
Candlestick Park	Ca	11,000	20	33,000	4,900	7,900	460	3,300	1,700
Candlestick Fishing Pier	Pi	13,000	800	11,000	1,300	110,000	13,000	13,000	500

Table 3-4. Coliform Survey Results, MPN/100 ml (continued)

Sampling stations		Date: 2/21/80 Time: 1035-1325		Date: 2/22/80 Time: 1110-1330		Date: 2/23/80 Time: 1250-1535		Date: 2/24/80 Time: 1350-1630	
Location	Code	Total	Fecal	Total	Fecal	Total	Fecal	Total	Fecal
Vicente Street	Vi	1,300	130	130	<20	40	<20	330	20
Ortega Street	Or	2,200	20	170	70	170	20	70	2
Lincoln Way	Li	13,000	1,700	230	50	130	<20	110	2
Kellys Cove	Kc	460	170	230	<20	80	<20	270	5
Lobos Creek	Lo	170,000	13,000	490	230	2,300	40	140,000	4,900
Bakers Beach	Ro	1,300	490	490	80	4,600	170	1,100	20
Fort Point	Fp	1,300	230	1,100	330	1,400	230	14,000	50
Crissy Field	Cf	3,300	490	1,300	130	790	230	4,900	110
Baker Street	Da	1,700	330	2,200	130	11,000	140	13,000	790
Fort Mason	Fm	790	230	1,400	170	7,000	220	23,000	20
Municipal Pier	Mp	2,300	230	4,900	230	11,000	330	4,900	130
Aquatic Park	Ap	1,700	230	2,300	230	4,900	310	33,000	20
Warmwater Cove	Ww	2,300	230	7,000	1,100	3,300	330	4,600	130
Candlestick Park	Ca	2,300	330	4,900	490	1,300	80	2,300	50
Candlestick Fishing Pier	Pi	70,000	7,900	1,300	490	790	130	1,300	23

Table 3-5. Days of Excessive Coliform Levels

Sampling station	>10,000 MPN/100 ml		>1,000 MPN/100 ml	
	Total	Fecal	Total	Fecal
Vicente Street	4	2	6	5
Ortega Street	2	0	5	2
Lincoln Way	5	3	6	5
Kellys Cove	4	0	6	4
Lobos Creek	7	6	9	7
Bakers Beach	2	1	9	3
Fort Point	2	0	9	2
Crissy Field	1	0	9	0
Baker Street	5	0	10	6
Fort Mason	2	0	8	2
Municipal Pier	2	0	10	2
Aquatic Park	2	0	9	2
Warmwater Cove	2	0	10	3
Candlestick Park	4	0	10	5
Candlestick Fishing Pier	7	4	9	6

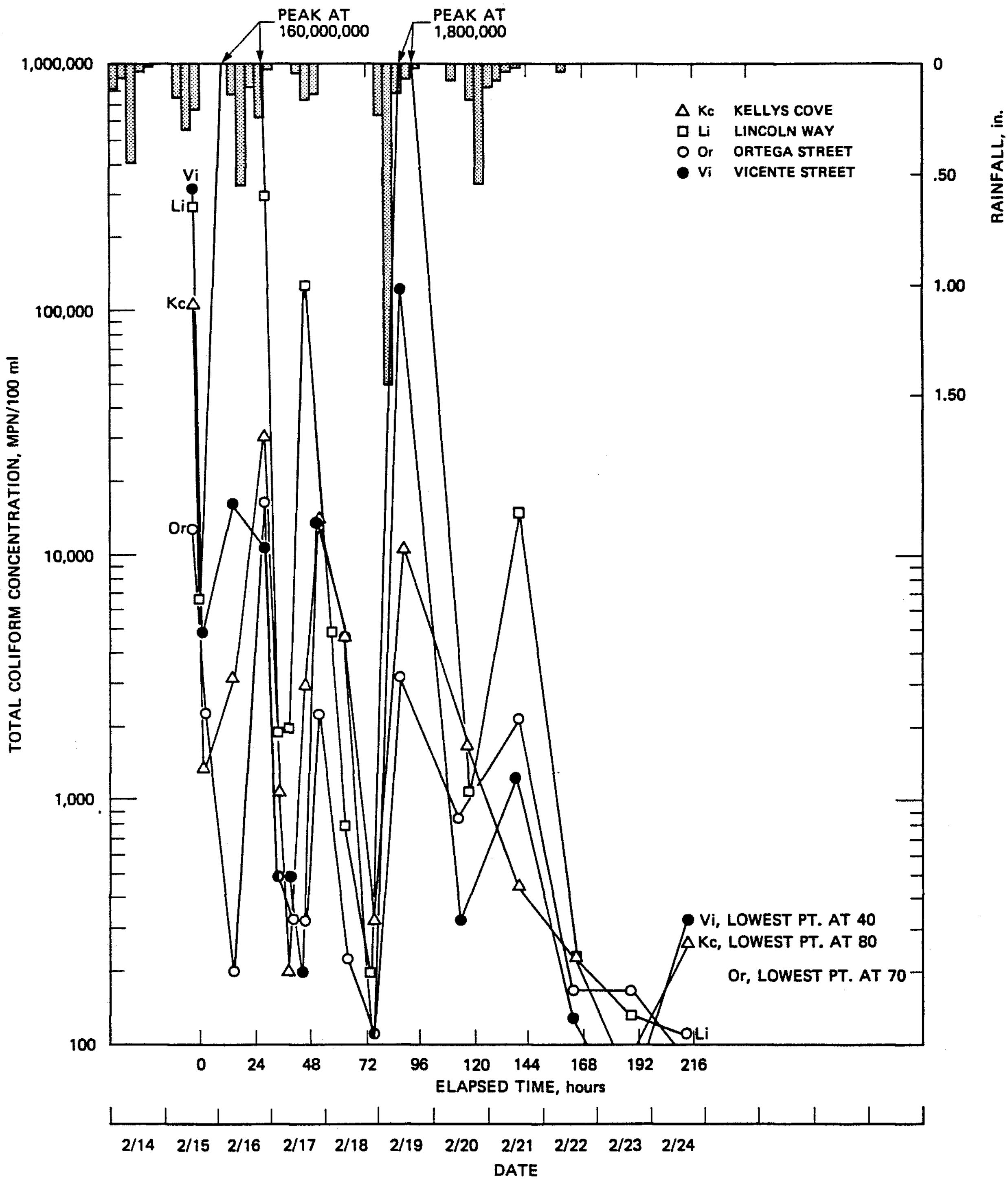


Figure 3-7 Ocean Beach Coliform Results

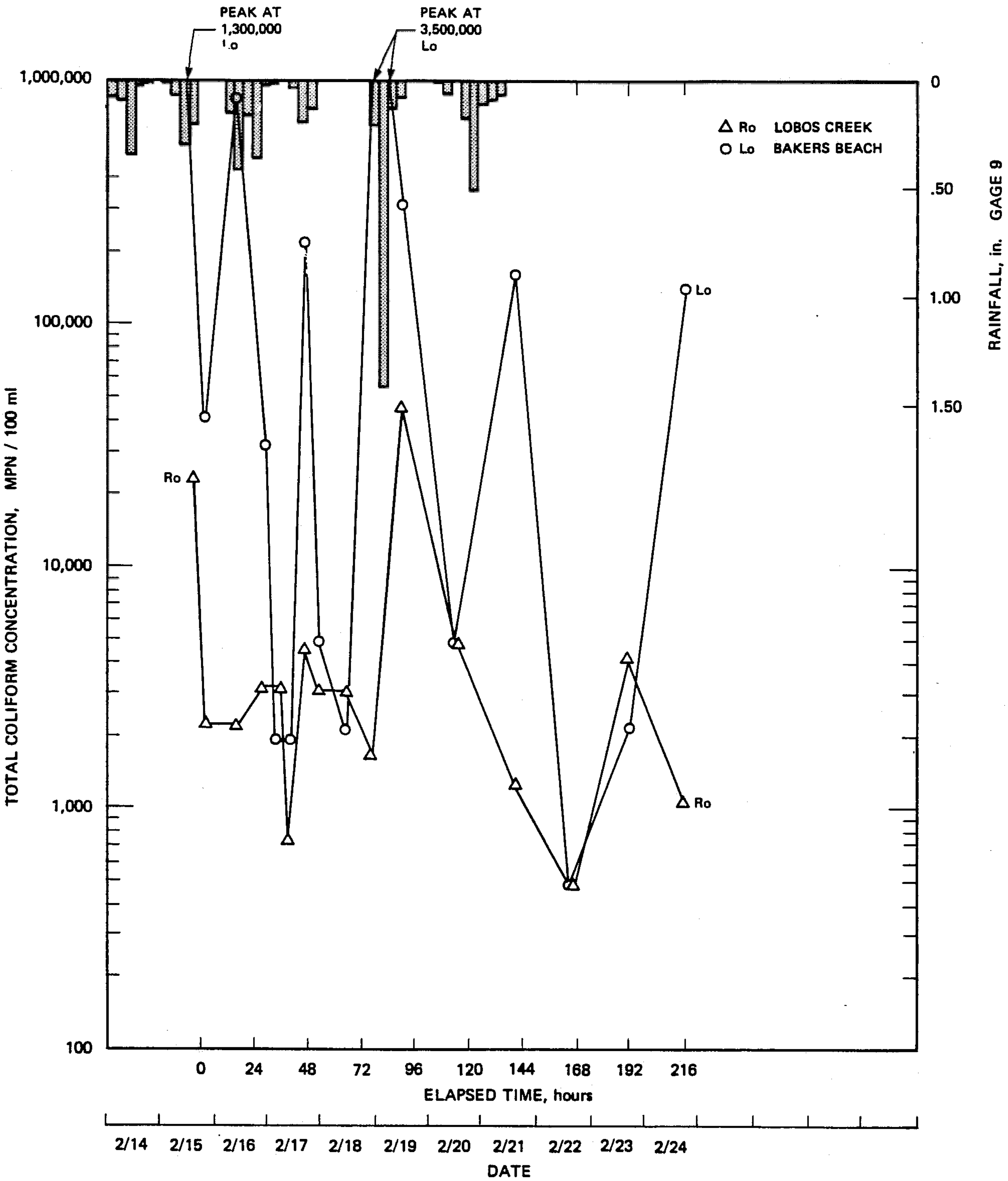


Figure 3-8 Phelan Beach and Baker Beach Coliform Results

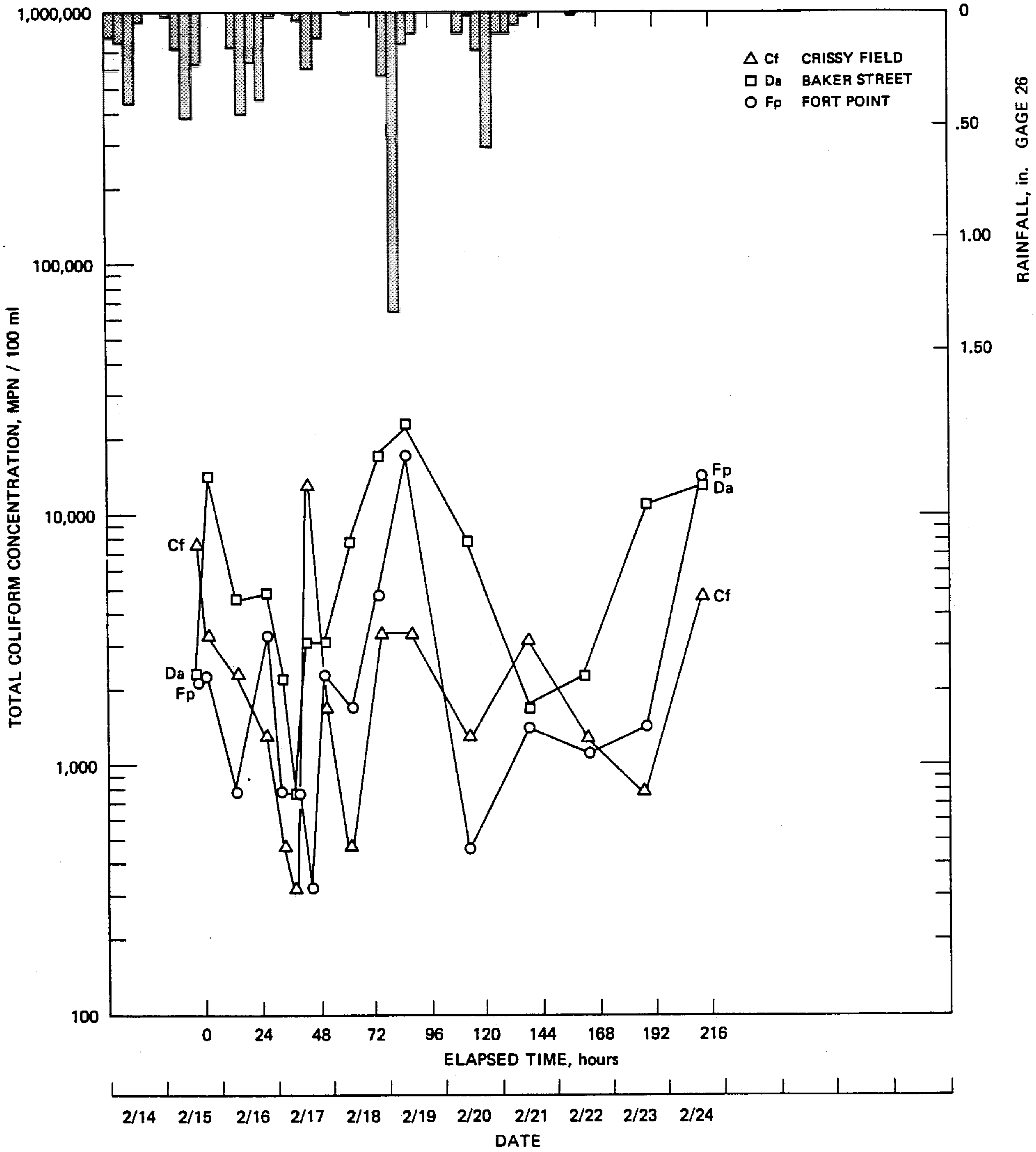


Figure 3-9 Fort Point to Baker Street Coliform Results

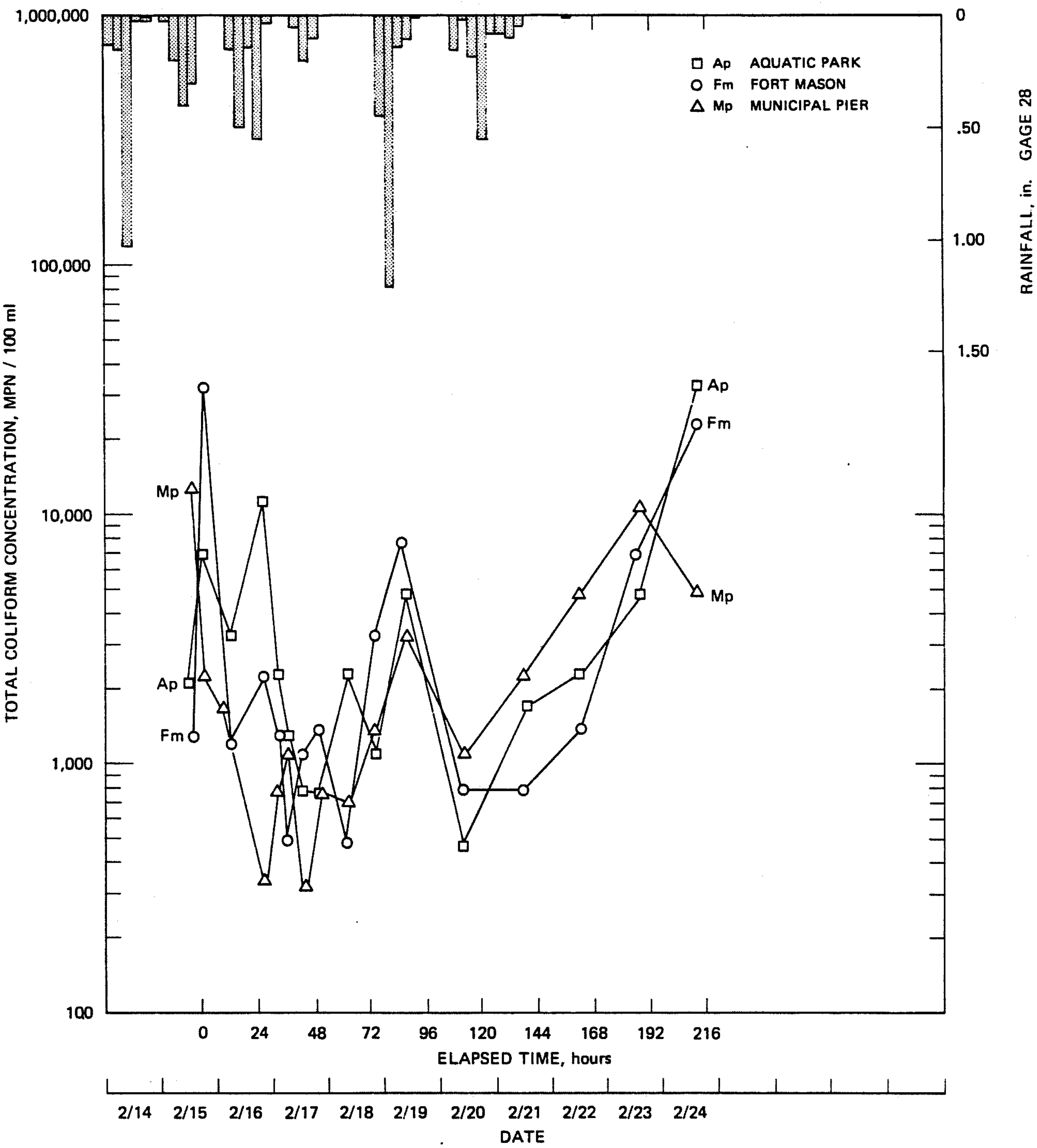


Figure 3-10 Aquatic Park Coliform Results

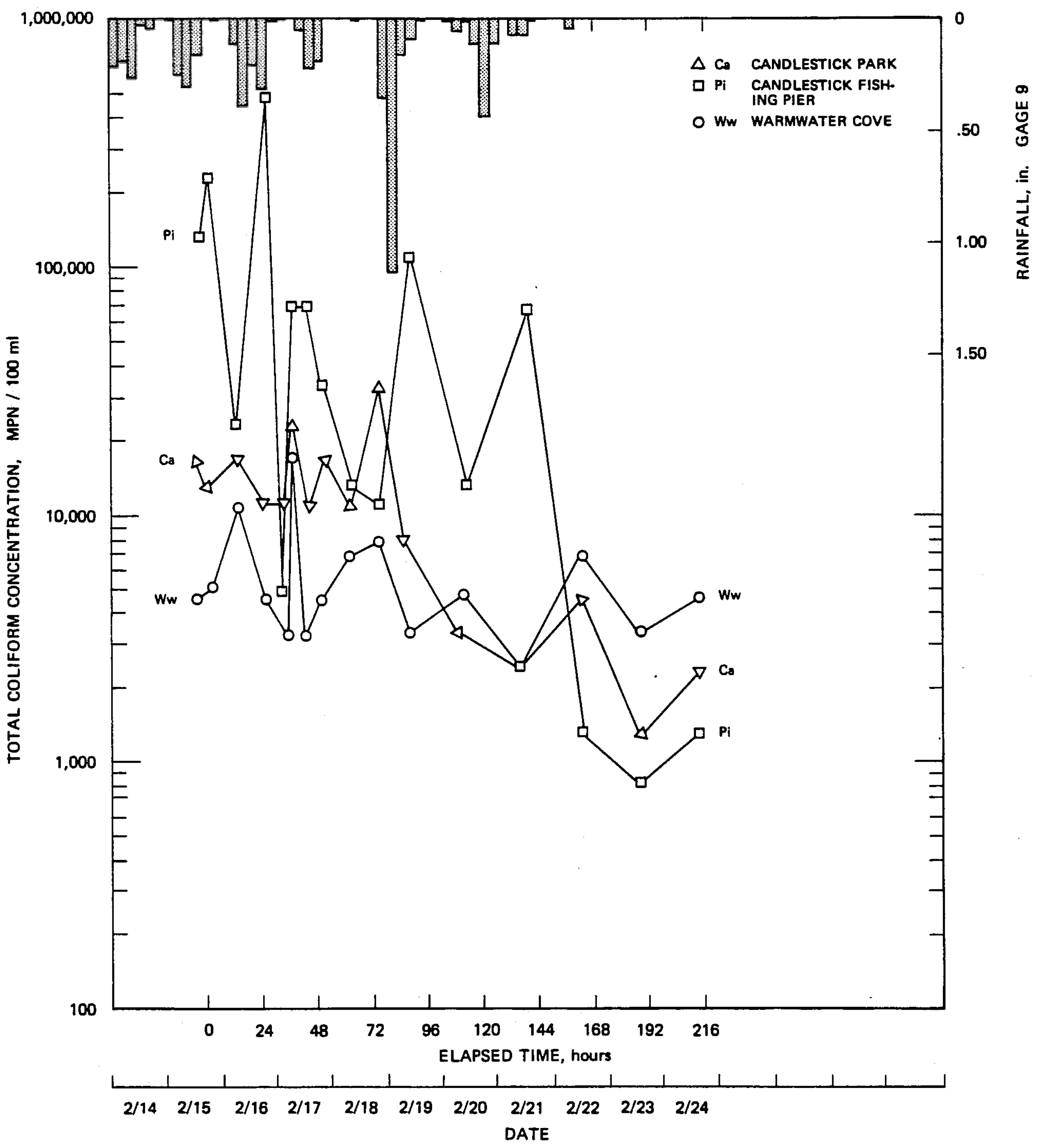


Figure 3-11 Southeast Coliform Results

Total rainfall amounts for the six storms were 5.35 inches at gage 7 on the west side, 5.64 inches at gage 9 near Lobos Creek, 6.18 inches at gage 26 in the Baker Street area, 6.62 inches at gage 28 near North Point, and 5.77 inches at gage 43 in the Yosemite area.

Rainfall amounts for the first two storms on February 14 and 15, 1980, ranged from 0.58 to 0.97 inches for the five gages selected. Peak hourly rainfall intensities were on the order of 0.15 inches per hour to 0.19 inches per hour, making these storms about the tenth largest storms of the year, on an average annual basis from return period-intensity-duration curves for the FOB city raingage. The third storm, occurring on February 16, 1980, had rainfall amounts ranging from 1.05 to 1.36 inches and peak intensities ranging from 0.22 inches per hour to 0.34 inches per hour making it about the eighth to the third largest storm of the year. The fourth storm was smaller with rainfall amounts ranging from 0.34 to 0.48 inches. Starting on February 18 and continuing to February 19, 1980, was the largest of the six storms. Rainfall amounts ranged from 1.84 to 1.91 inches with peak intensities of 0.44 to 0.53 inches per hour. Consequently, this storm had a return period of 1.3 to 2.0 years. The last storm started on February 20, 1978, and ended on February 21, 1978. Rainfall amounts ranged from 0.95 to 1.21 inches and peak intensities ranged from 0.21 to 0.20 inches per hour which makes this storm similar to the third storm.

Comparing the rainfall data with the total coliform data shows a direct relation between the peak rainfall intensities and the peak coliform concentrations. Coliform concentrations peaked five times corresponding to the five storms during the sampling period; concentrations were already elevated from the first storm when sampling began. Usually the highest peak coliform concentration followed the largest storm on February 18 to 19, 1980. This peak was five to ten times higher at most stations than the response to the other storms in the sequence.

Coliform concentrations declined rapidly after the rainfall ceased. Referring to Figures 3-7 through 3-11, the time for 90 percent decay from a given value, commonly called the T_{90} value, is on the order of 12 hours observed. This decay is due to the combined effects of die away and dilution. Generally, following a peak value, the coliform concentration dropped to below 1,000 MPN/100 ml on the day after the peak concentration was measured. Because each of these storms had a duration less than 1 day, the total coliform concentrations were elevated above 1,000 MPN/100 ml for only 2 days. However, it appears reasonable to continue to assume that each combined sewer overflow under the proposed master plan will cause total coliform concentrations to exceed 1,000 MPN/100 ml for about 3 days.

Figures 3-12, 3-13, and 3-14 show the percent number of days that observed fecal concentrations equaled or were below a specified value. The data include only one sampling per day, which was conducted at slack water before flood tide on February 15 through 24. For example, on Figure 3-12, on 50 percent of the days (5 days out of a possible total of 10), the fecal concentration at Lobos Creek was 2,000 MPN/100 ml or less.

For the ocean side, shown on Figure 3-12, median concentration over the 10-day period ranged from 20 MPN/100 ml at Ortega Street to 2,000 MPN/100 ml at Lobos Creek. Concentration was 50 MPN/ 100 ml or less 26 to 36 percent of the days (time) at all ocean sampling stations except at Lobos Creek where the frequency was 12 percent of the days. A possible explanation for the higher Lobos Creek values is that land surface drainage which was not intermittent during the period, as were the overflows, also flows through Lobos Creek.

For the north bay side areas, shown on Figure 3-13, median fecal concentration was 200 MPN/100 ml, which was less than half of the oceanside average median value. Concentration was 50 MPN/ 100 ml or less 20 to 25 percent of the days at all stations except Baker Street (near the outfall) where the frequency was 10 percent. Overall average scope of the curves over the 10-day period was slightly flatter for the north bay side than for the ocean side, indicating higher average background concentrations. Comparing total coliform concentrations and fecal coliform concentrations for the last 2 to 3 days of the sampling period for all six North Shore area stations shows high total and low fecal coliform concentrations. This is probably due to high delta outflow and not to combined sewer overflows during this period.

On Figure 3-14, median concentration ranged from 330 to 800 MPN/100 ml at the southeast bay side, higher on the average than the ocean side. Concentration was 50 MPN/100 ml or less on 15 to 20 percent of the days except at Warmwater Cove, where observed fecal coliform concentration was not less than 80 MPN/ 100 ml throughout the survey period. Perhaps this concentration is due to the continual renewal of water provided to the cove by the cooling water intake from the main tidal stream in the bay.

PREVIOUS FIELD STUDY RESULTS

Field studies similar to those conducted in this study have been done previously by city staff and consultants. These include the City of San Francisco's public health daily monitoring, CH2M HILL's 1979 studies and work done by Brown and Caldwell, also in 1979. They provide additional insight into the dilution of combined sewer overflows provided by bay and ocean waters. The results from these studies are summarized below.

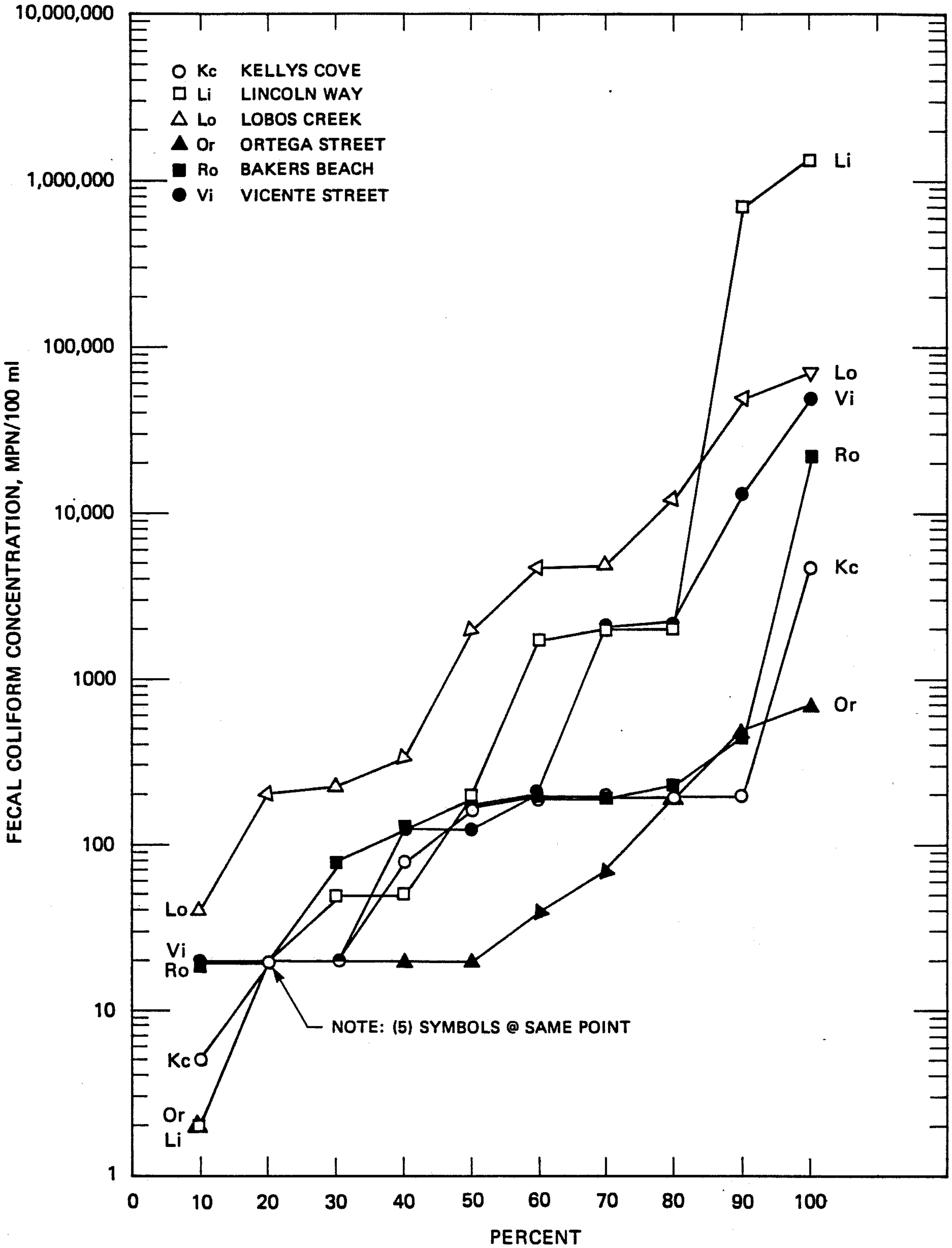


Figure 3-12 Percent Number of Days That Observed Fecal Concentration was Equal to or Below Specified Value for Westside Stations

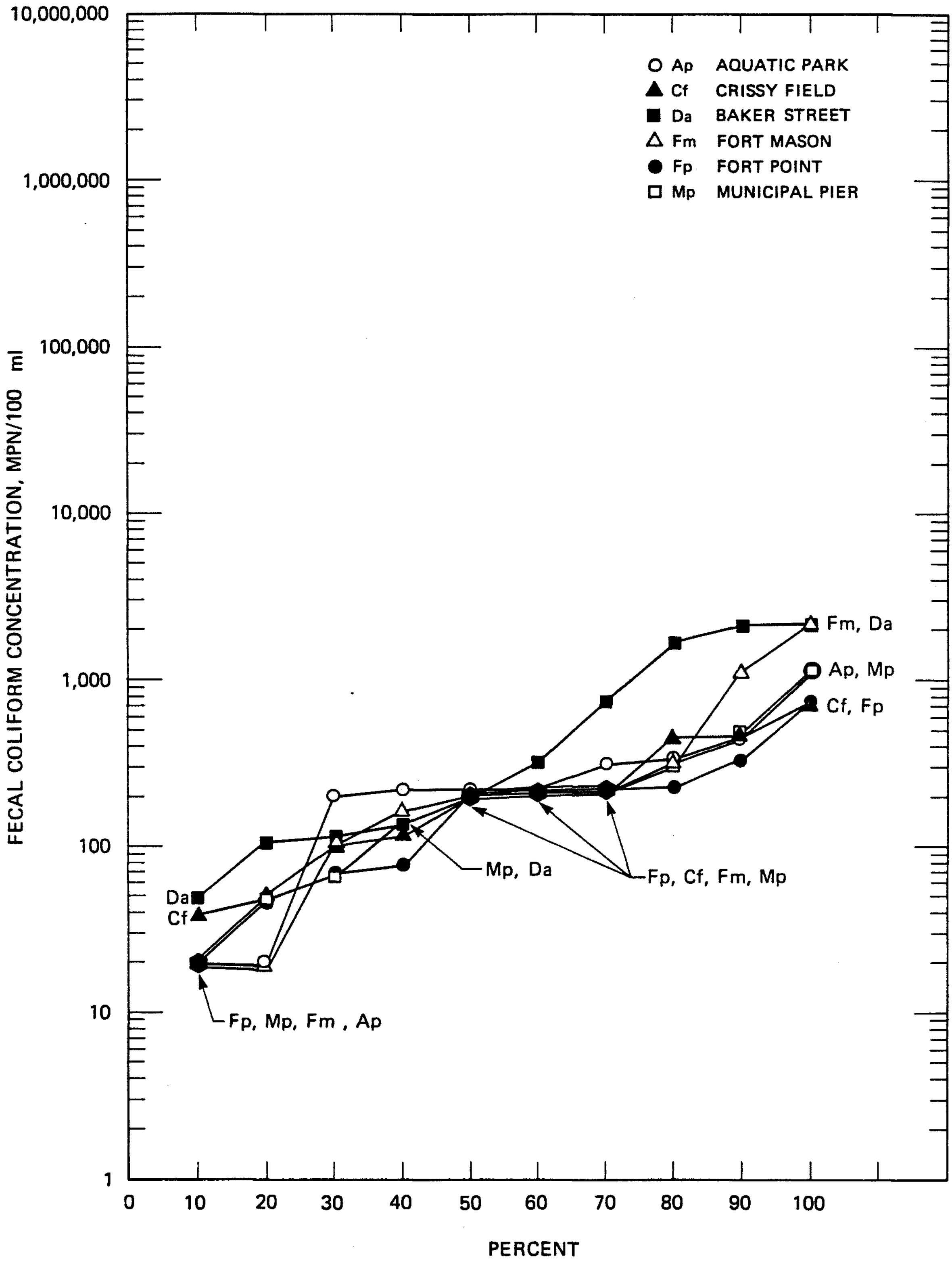


Figure 3-13 Percent Number of Days That Observed Fecal Concentration was Equal to or Below Specified Value for North Shore Stations

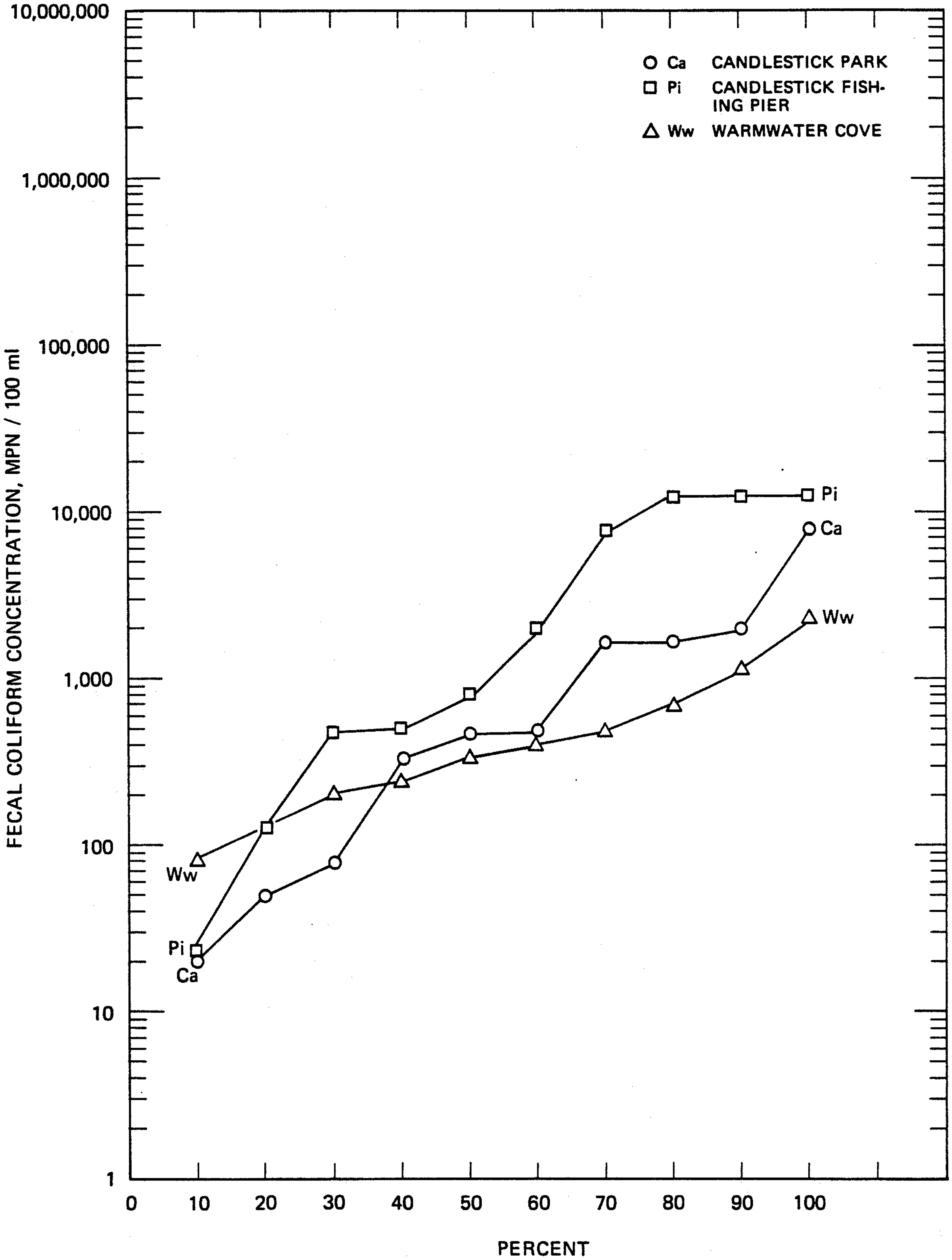


Figure 3-14 Percent Number of Days That Observed Fecal Concentration was Equal to or Below Specified Value for Southeast Stations

City of San Francisco Daily Bacteriological Monitoring

This bacteriological monitoring program has been in effect since November 1972. Since that time, coliform concentrations in the receiving waters at locations shown on Figure 3-15 have been determined on a daily basis, Monday through Thursday, by personnel from the three sewage treatment plants. Sampling is dependent upon the accessibility of the station, which varies due to tidal conditions and because some of the piers in the North Point area are occasionally locked. Grab samples taken at the shoreline are analyzed for total coliforms. The determination, presumptive and confirmed, is made on a three-decimal tube dilution basis, and reported as MPN/100 ml. Observations of visible pollution along the shoreline and in the water have been noted. The collected data are analyzed by the Regional Board and the City Department of Public Health; the latter agency also conducts supplemental water quality surveys to assure compliance with beach water standards.

Shown in Table 3-6 are selected coliform data from the monitoring program taken during isolated storms. Generally, the data indicates an immediate response to an overflow and a 3- to 4-day subsiding period. Variance from this pattern can be explained by changing tides which may carry a wastewater plume into and out of the area. Also, tidal conditions at the time of the overflow can effect the initial movement and dispersion of a plume. In addition, differing storm durations can result in various coliform levels. Coliform data from 1972 to 1977 were analyzed by J. B. Gilbert and Associates and published in the report entitled "Effects of Combined Sewer Overflow on Receiving Water Quality" in August 1978.

An extensive water quality monitoring program performed by CH2M HILL was described in their 1979 report entitled "Bayside Overflows." During this project, offshore and nearshore grab samples were collected before, during, and after three overflow events to determine water quality characteristics. There were 15 offshore stations and 8 nearshore stations. On the first 2 days of the overflow event, samples were collected at each offshore and nearshore station every quarter tidal cycle at slack water. On the following 3 days, samples were collected at one high and one low slack water condition during daylight hours. Each water sample was analyzed for total and fecal coliforms, suspended solids, and conductivity. When water depths were greater than 15 feet, samples were collected 2 feet below the surface and at 6 feet above the bottom. When water depths were less than 15 feet, samples were collected 2 feet below the surface.

The background preoverflow levels of coliforms ranged from 200 to 500 MPN/100 ml while fecal coliform levels were 30 to 200 MPN/100 ml. By comparison, dry weather total coliform levels reported for 1978 were usually less than 100 MPN/100 ml while fecal coliform levels were less than 50 MPN/100 ml.

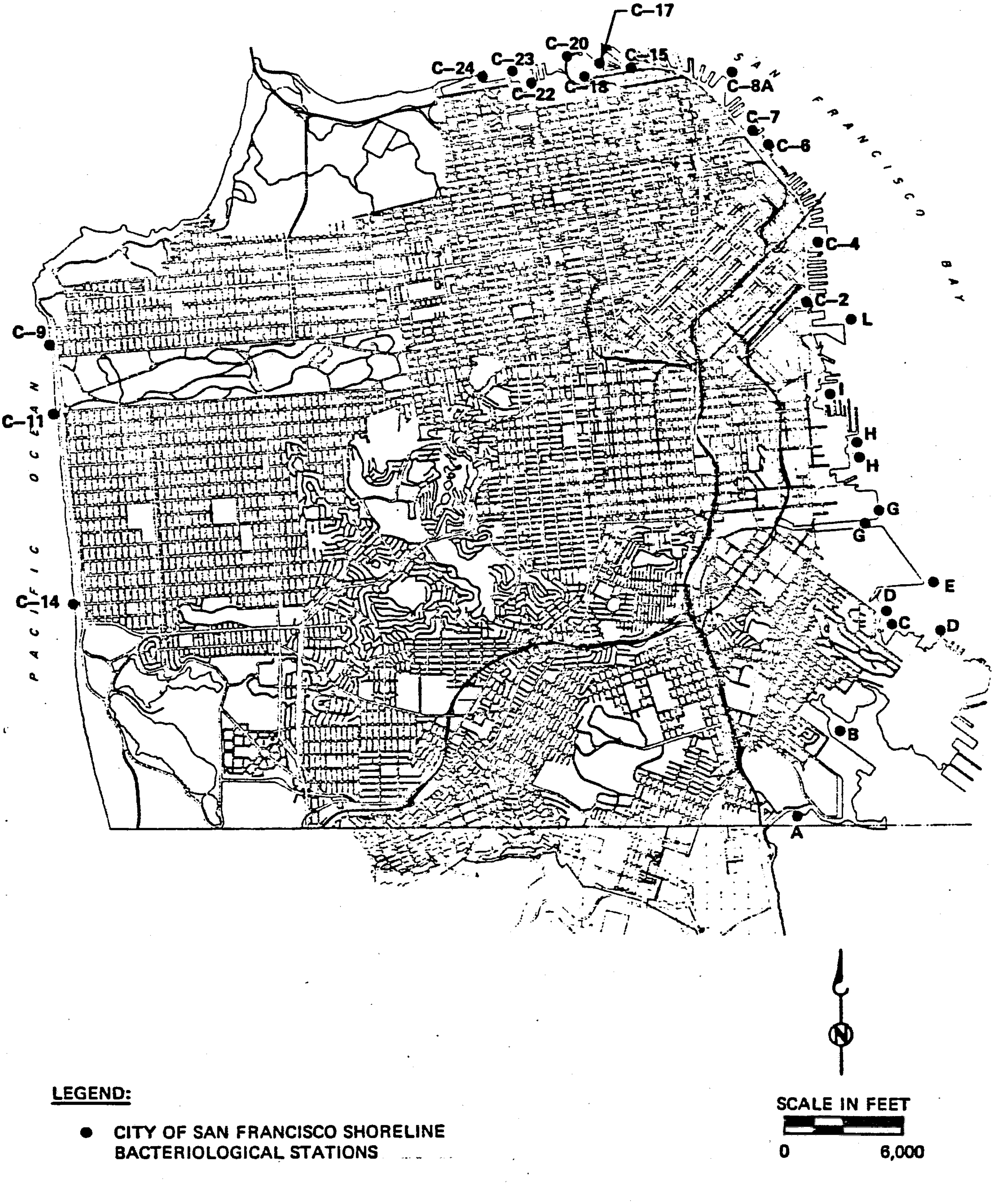


Fig. 3-15 City of San Francisco Shoreline Bacteriological Stations

Table 3-6. Selected Coliform Data From City of San Francisco Sampling Program

Station	Storm date	Rainfall, in.	Total coliform, MPN/100 ml				
			Day 0	Day 1	Day 2	Day 3	Day 4
Richmond Sunset District							
C-9	7/8/74	.57	≥24,000	620	230	230	-
C-11			≥24,000	2,400	620	45	-
C-14			11,000	7,000	500	60	-
C-9	12/2/74	.60	≥24,000	≥24,000	930	430	-
C-11			2,400	11,000	2,400	750	-
C-14			≥24,000	≥24,000	930	24,000	-
C-9	2/29/76	1.41	-	4,600	24,000	430	-
C-11			-	11,000	≥24,000	4,600	-
C-14				390	11,000	2,400	
North Point District							
C-2	2/18/75	.75	230	2,290,000	4,600	-	-
C-4			230	46,000	2,300	-	-
C-2	9/19/77	.71	≥240,000	110,000	24,000	760	-
C-4			24,000	4,200	24,000	810	-
C-2	11/21/77	1.53	≥240,000	≥240,000	7,600	-	-
C-4			≥240,000	110,000	1,500	-	-
C-2	11/24/78	.98	430	≥240,000	≥240,000	230	-
C-4			430	1,000	230	4,600	-
C-6	9/25/72	.52	130	230,000	2,400	2,400	-
C-7			46	2,300	2,400	2,300	-
C-6	7/8/74	.61	23,000,000	2,300	620	230	-
C-7			2,300	2,300	620	230	-
C-6	9/19/77	.71	24,000	4,600	2,400	940	-
C-7			110,000	4,600	430	430	-
C-22	12/2/74	.52	-	11,000,000	2,400	2,300	-
C-23			4,300	430	430	-	-
C-24			-	460,000	240	430	-
C-22	9/19/77	.71	110,000	930	1,500	430	-
C-23			≥24,000	230	90	140	-
C-24			46,000	2,400	430	230	-
C-22	4/24/78	.98	230	230	4,600	2,100	-
C-23			40	430	90	230	-
C-24			430	4,600	230	4,600	-
C-22	12/17/78	.44	15,000	930	2,400	760	-
C-23			2,400	930	630	930	-
C-24			4,600	1,500	1,200	430	-
C-15	9/25/72	.52	60	2,300	2,400	2,300	-
C-17			≤30	2,300	940	2,300	-
C-18			-	-	2,400	2,300	-
C-20			-	-	4,600	620	-

Table 3-6. Selected Coliform Data From City of San Francisco Sampling Program (continued)

Station	Storm date	Rainfall, in.	Total coliform, MPN/100 ml				
			Day 0	Day 1	Day 2	Day 3	Day 4
North Point District (continued)							
C-8A	7/8/74	.61	2,300	230	24,000	-	-
C-15			930	430	930	430	-
C-17			6,200	2,300	430	130	-
C-18			620	6,200	150	5,000	-
C-20			2,300	620	760	60	-
C-15	2/18/75	.63	4,300	4,600	2,300	-	-
C-17			90	4,600	2,300	-	-
C-18			430	230	430	-	-
C-20			230	24,000	430	-	-
C-8A	9/19/77	.71	11,000	1,500	230	930	-
C-15			23,000	430	4,600	930	-
C-17			150	230	230	40	-
C-18			430	230	430	90	-
C-20			46,000	930	230	90	-
South East District							
H'	7/8/74	.48	6,200	500	230	60	-
H'	2/29/76	1.20	-	930	2,400	430	230
H	9/19/77	.72	≥24,000	11,000	-	-	-
I			≥24,000	4,600	-	-	-
L			≥24,000	2,400	-	-	-
H	11/21/77	1.53	≥24,000	11,000	4,600	-	-
I			11,000	2,400	750	-	-
L			4,600	2,400	150	-	-
G'	3/19/73	.68	23	2,300	≥240,000	2,300	-
G			230	2,300	13,000	230	-
G'	7/8/74	.53	7,000,000	2,400,000	2,300	620	-
G			2,400,000	23,000	2,300	620	-
G'	12/2/74	.61	240,000	23,000	93,000	23,000	-
G			240,000	430,000	9,000	9,300	-
G'	3/15/77	1.33	≥24,000	≥24,000	4,600	-	-
G			≥24,000	≥24,000	2,300	-	-
C'	10/7/73	.61	-	-	7,000	230	2,300
D'			-	-	24,000	620	13,000
C'	7/8/74	.47	60	6,200	620	≤23	-
D'				6,200	2,300	620	-
C'	2/29/76	1.18	-	2,400	150	90	≤30
D'			-	4,600	230	2,300	230
C'	3/15/77	1.14	2,300	4,600	2,300	-	-
D'			-	4,600	640		

Table 3-6. Selected Coliform Data From City of San Francisco Sampling Program (continued)

Station	Storm date	Rainfall, in.	Total coliform, MPN/100 ml				
			Day 0	Day 1	Day 2	Day 3	Day 4
South East District (continued)							
C'	11/21/77	1.53	11,000	2,400	1,500	-	-
D'			4,600	930	150	-	-
D			24,000	4,600	2,100	-	-
E			2,100	2,400	430	-	-
A	10/9/72	4.87	-	≥240,000	2,400,000	625,000	-
A	3/19/73	.64	-	23,000	23,000	2,300	-
A	12/2/74	.59	24,000	24,000	4,300	3,900	-
A	1/6/75	1.10	46,000	7,500	3,900	-	-
A	11/21/77	1.53	≥24,000	1,500	11,000	-	-
B			-	≥24,000	≥24,000	-	-
A	3/21/78	.28	430	24,000	930	-	-
B			1,500	≥24,000	3,600	-	-
A	4/23/79	.42	≥24,000	230	930	-	-
B			≥24,000	-	11,000	-	-

As expected, the coliform levels in Islais and Channel Creeks after an overflow indicate that there is a direct correlation between combined sewer overflows and coliform levels. Following the overflow sampled, the coliform levels in the channels usually rose by about four orders of magnitude over the background levels. When the overflow stopped, the coliform levels began dropping until another overflow occurred, at which time levels rose again approximately one order of magnitude and then decreased until the next overflow. The last overflow in the sampling period occurred during the second day. This was followed by a short lag time and then the coliform levels began decreasing rapidly. These decreasing coliform levels can be attributed both to the natural die-off rates and the physical process of dilution and sedimentation. Normal levels were generally found within 48 hours after the last overflow. The T_{90} value is approximately 24 hours in Islais Creek and Channel. In the open bay, the T_{90} value is even smaller mainly due to the increased dispersion and advection.

The analyses also showed a decreasing coliform gradient moving out into the bay from each overflow structure. At the end of the pier line, the initial coliform concentration was diluted by at least 10:1. In the offshore stations, the coliform concentration was generally about 2,000 MPN/100 ml, or one order of magnitude over the measured background level within 2 days.

CH2M HILL's study indicated that water quality impacts of combined sewer overflows (CSO) were mainly confined to nearshore areas, particularly within Channel and Islais Creek. Offshore effects were minimal and generally could not be directly attributable to the CSOs. Most impacts at offshore stations were attributable to general rainstorm effects.

Brown and Caldwell's 1979 Studies

The Brown and Caldwell field program, as discussed in the report entitled "Bayside Wet Weather Facilities Revised Overflow Control Study," was initiated in response to the request from the U.S. Environmental Protection Agency for data on the toxic constituents of overflows. Levels of lead, mercury, cadmium, TICH, and stickleback survivals at specific overflow structures were measured. The City elected to add total coliform, fecal coliform, pH, temperature, and salinity sampling in order to gain insight into the dispersion of overflow plumes. The results were tabulated in Appendix B of the Bayside Wet Weather Facilities Revised Overflow Control Study and are discussed below.

The coliform levels increased, as expected, in response to overflows. An overall view of the data showed that concentration of coliforms were high near the outfalls and decreased with distance from the outfall. The Lincoln Way Outfall showed a typical dispersion pattern. Total coliform levels of approximately

10,000/100 ml were noted up to 1,000 feet north and south of the outfall. The coliform data showed levels up to 1 million/100 ml near the Bakers Beach Outfall during overflows, and 10 to 100 times less at the station 2,000 feet northeast of the outfall. In the South Basin Canal, coliform levels at the point of the outfall and 1,500 feet downstream reached 1 million/100 ml. The data for a station 5,000 feet from the outfall indicated orders of magnitude 3 to 4 times less than the numbers at the outfall. This decrease could be due to dilution or because the overflow plume may not have reached the station by the time the one sample was taken.

CHAPTER 4

EVALUATION OF OVERFLOW DISCHARGE LOCATIONS AND PRIORITIES

When overflows occur from the transport/storage system, they will be discharged to bay and ocean waters. The overflows could be accommodated in existing outfalls or in new outfalls constructed to improve dilution capabilities. This section describes the overflow discharge alternatives available for each outfall consolidation project. Prioritizing the discharge from existing outfalls is emphasized although the cost-effectiveness of extending outfalls to improve dilution capability is also described.

FUTURE OVERFLOW CHARACTERISTICS

Following the completion of outfall consolidation projects throughout the City, the frequency, duration, and quantity of overflows will change. The San Francisco MAC (SFMAC) computer model is being used to plan future transport/storage facilities. Output from 70-year simulations is summarized in Table 4-1. Overflow durations average between 1 and 4 hours. Overflow quantities range from about 2 million gallons to over 40 million gallons. Overflow durations are longer and quantities larger in the areas where wet weather flow is being aggregated, i.e, Channel and Islais Creek, and because the immediate tributary area is larger.

The impact of these overflows on beneficial use areas will depend upon the dilution in the receiving water between overflow discharge location and the beneficial use area. The float survey results, discussed previously, can be used to give an indication of impact on selected beneficial use areas from specific overflow discharge locations. For example, if all of the overflow from the North Shore area could be discharged through the Jackson Street Outfall, then it would be diluted by a factor of 100 when it reached the Aquatic Park. This diluation is based upon several days of accumulation at the Aquatic Park from Jackson Street and is representative more of floatable material than other dissolved and suspended material in the overflow.

REQUIRED OUTFALL CAPACITY

The required outfall capacity for combined sewer overflows is dependent upon the size of the tributary drainage area, the allowable number of overflows, the rainfall characteristics, and

Table 4-1. Future Overflow Characteristics

Consolidation project	Number of overflows per year	Duration,[a] hr		Quantity,[a] MG	
		Per year	Per event	Per year	Per event
West Side	8	90.0	11.2	459.7	57.5
North Shore	4	13.2	3.3	57.2	14.3
Channel	10	40.5	4.1	407.6	40.8
Islais Creek	10	32.4	3.2	380.4	38.0
Yosemite	1	1.4	1.4	3.5	3.5
Sunnydale	1	1.1	1.1	2.2	2.2

[a]Average per year and per event based upon SF MAC model computer simulations.

the transport/storage capacity available when peak rainfall intensities occur. Shown in Table 4-2 is the relation between rainfall intensity and occurrences per year. For the Channel and Islais Creek areas, where the National Pollutant Discharge Elimination System (NPDES) permit allows ten overflows per year, the smallest overflow of the year would probably result from a storm with a maximum rainfall intensity of 0.17 inch per hour for 1 hour duration. There would be nine storms larger than this in an average year. The largest storm of the year, on the average, will have a maximum rainfall intensity of 0.43 inch per hour and this would probably produce the annual maximum overflow rate. There will be, of course, larger storms, occurring less frequently, which will produce even larger overflows. Return period intensities for storms occurring once every other year and once in 5 years are also tabulated in Table 4-2.

A preliminary estimate of the required outfall capacity to accommodate the storms having rainfall intensities, tabulated in Table 4-2, are shown in Table 4-3 for each outfall consolidation project. The required capacities shown been estimated as follows. First, the City average rainfall intensities in Table 4-2 were multiplied by rainfall adjustment factors, which range from 0.83 to 1.18, to account for area rainfall variability. Then, making a conservative assumption that storage basins are full when the maximum rainfall intensity occurs, the intensity is multiplied by the runoff coefficient and the drainage area to predict peak runoff using the rational method. Finally, subtracting the proposed pumping or treatment rate for the outfall consolidation project gives the required total outfall capacity for each project. Referring to Table 4-2, the Channel area, for example, would require 275-million-gallon-per-day (mgd) outfall capacity to handle the tenth largest storm of the year and up to 980-mgd capacity for the largest storm of the year. Once every 5 years, 1,491 mgd would be required.

The estimates of required outfall capacity in Table 4-2 are preliminary because they do not account for the amount of available treatment and storage capacity when the peak runoff occurs. The required outfall capacity for the Richmond Transport system was checked using available output data from the SFMAC computer program (70 years of simulated overflows) for various return period storms. The estimates shown in Table 4-2 were found to be 20 to 25 percent too high. Thus, storage available at the time the peak runoff occurred was able to attenuate the peak by this amount. Consequently, the required outfall capacities may also be overestimated for the remainder of the City where SFMAC computer output of this type is not available at this time. During Bayside Facilities Planning, the estimates of required outfall capacity will be refined. Reducing the required outfall capacity will make it more feasible and more beneficial to prioritize overflow locations.

Table 4-2. Maximum Rainfall Intensity Equaled or Exceeded Stated Number of Times Per Year

Occurrences/yr	Maximum rainfall intensity, in./hr
0.2	0.62
0.5	0.52
1	0.43
2	0.37
4	0.28
8	0.23
10	0.17

Source: Metcalf & Eddy, Inc., "Southwest Water Pollution Control Plant Project," Project Report Figure 4-7, September 1979.

Table 4-3. Preliminary Estimate of Required Outfall Capacity, mgd

Area	Frequency of outfall capacity being equalled or exceeded, events per year						
	0.2	0.5	1	2	4	8	10
Westside							
Lake Merced	472	387	310	259	182	139	-[a]
Westside Transport	992	811	648	539	377	286	-[a]
Richmond Transport	422	344	274	227	157	118	-[a]
Total, Westside	1,886	1,542	1,234	1,025	716	543	-[a]
North Shore							
Marina	289	236	188	156	109	-[a]	-[a]
Beach	111	91	73	61	43	-[a]	-[a]
Jackson	219	179	144	120	84	-[a]	-[a]
Total, North Shore	619	506	405	337	236	-[a]	-[a]
Channel	1,472	1,199	957	797	555	419	258
Mariposa	43	32	23	16	7	1	-
Islais Creek	1,388	1,148	932	788	572	452	308
Hunters Point	4	2	-	-[a]	-[a]	-[a]	-[a]
Yosemite	217	166	120	-[a]	-[a]	-[a]	-[a]
Sunnydale	106	66	31	-[a]	-[a]	-[a]	-[a]
Total Bayside	3,868	3,142	2,491	1,960	1,391	895	583

[a]Accommodated by transport/storage system; no overflow.

WESTSIDE OVERFLOWS

In the following paragraphs, the most favorable locations for overflows will be established for the west side followed by alternatives to achieve 10:1 dilution and/or prioritize discharges from existing outfalls. Recommendations are based upon cost-effectiveness.

Most Favorable Locations for Overflows

Specified beneficial uses (NPDES Permit No. CA0038415) on the west side between Lake Merced and Bakers Beach Outfalls (1 through 8) are water-contact recreation, nonwater-contact recreation, marine habitat, ocean commercial and sport fishing, fish migration, and wildlife habitats. Beneficial uses are summarized on Figure 4-1. Clearly, the west side beneficial uses are extensive. The principal difference between existing beneficial areas is public access. Near the outfalls, shown on Figure 4-1, public access is restricted by high cliffs for the Mile Rock (Lands End) area and the Lake Merced area, the latter which also has limited parking. This limits the impacts of overflow in these areas. Shown in Table 4-4 are estimates of winter beach usage for the west side. Public beach use is highest at the Lincoln Way and Bakers Beach area. Public beach use near Vicente Street is lower than Lincoln Way. Based upon principal beneficial uses and public use, the most favorable overflow discharge locations are:

1. Lake Merced (Outfall No. 1).

2. Mile Rock (Outfall No. 4 and present outfall for Richmond-Sunset Water Pollution Control Plant).

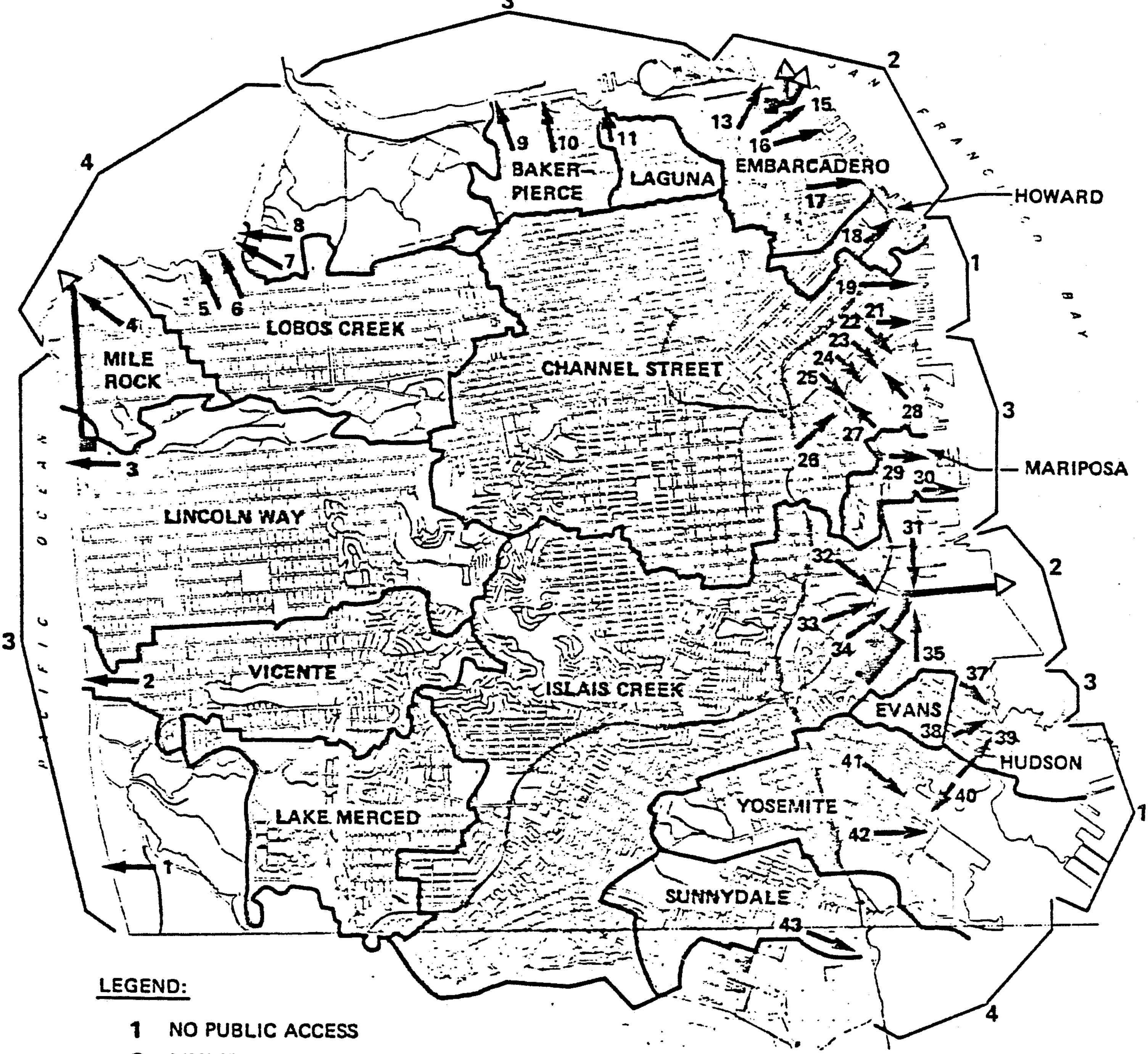
3. Sea Cliff (Outfall No. 6).

4. Vicente Street (Outfall No. 2).

It follows that the Lincoln Way and Bakers Beach Outfalls are less favorable locations than the above for discharging overflows.

Alternatives

In order to comply with the 10:1 dilution requirement, outfalls would probably have to be extended on the west side. From a practical standpoint, outfalls would have to be extended beyond the surf zone or about 3,000 feet. The City has previously estimated the cost of an outfall for westside overflows in the report entitled "Westside Wet Weather Facilities Revised Overflow Control Study," submitted to the California Regional Water Quality Control Board (RWQCB) in December 1978. A 3,000-foot-long, 15-foot-diameter outfall extension to the Lincoln Way overflow



LEGEND:

- 1 NO PUBLIC ACCESS
- 2 LIMITED PUBLIC ACCESS— NONWATER-CONTACT RECREATION
- 3 WATER-CONTACT RECREATION
- 4 SHELLFISH PROPOGATION FOR HUMAN CONSUMPTION
- TREATMENT PLANT
- DRAINAGE BOUNDARY
- SEWAGE TREATMENT PLANT OUTFALL
- COMBINED SEWER OVERFLOW

SCALE IN FEET

0 6,000

Fig. 4-1 Combined Sewer Overflow Location and Beneficial Use Areas

Table 4-4. Westside Beach Activity Survey

Activity	Estimates of daily winter time usage[a]									
	Baker Beach	Phelan Beach	Lands End	North of Fulton	Fulton to Lawton	Lawton to Santiago	Santiago to Sloat	Ft. Funston	Thornton Beach	Totals[b]
Swimming	<5	<10	-[c]	<5	<5	<5	<5	<5	<5	25 - 50
Surfing	<5	<5	-[c]	30	10	15	25	<5	-[c]	90
Fishing	20	5	10	-[c]	-[c]	6	5	5 - 10	5	60
Shell fishing	-[c]	<5	-[c]	-[c]	-[c]	-[c]	-[c]	-[c]	-[c]	-[c]
Wading below waist	15	5	-[c]	30	25	20	15	5	5	120
Wading above waist	<5	<5	-[c]	5	5	<5	<5	<5	<5	25
Noncontact usage	250	60	50[d]	600	430	220	260	300	35	2,165

[a]Based on Wastewater Program, December 1978 surveys.
[b]Less than 5 counted as 2-1/2 for total.
[c]Dash (-) indicates negligible.
[d]Considers only people on the several small pocket beaches in this area.

Source: West Side Wet Weather Facilities Revised Overflow Control Study, December 1978.

structure would be adequate for all but the peak 1-hour storm. To this outfall would be added a 660-foot diffuser with four multiport risers. This outfall would provide at least 10:1 dilution in all but a few extreme worst cases. The cost of this outfall was estimated to be $36 million (October 1978 dollars). Costs are approximately 12 percent higher today than when this estimate was prepared.

The other alternative is to utilize existing outfalls according to the above priorities. Existing outfall capacity, when connected to the proposed transport/storage systems, will have, generally, less capacity than at present. Estimates of outfall capacity, tabulated in Table 4-5, assume that overflow weirs will be set at +50 for Richmond Transport, +6 for Westside Transport, and +4 for Lake Merced Transport. Available head is the difference between these elevations and mean higher high water, -5.5, less head losses.

Matching the required outfall capacities (Table 4-3) with the available outfall capacity and recognizing the priorities for discharge listed above, shows that the existing Lake Merced Outfall is adequate for up to the 5-year storm for this area; the Mile Rock and the Vicente Outfalls are more than adequate for the largest storm of every 2 years for the west side system. Similarly, the Sea Cliff Outfall can handle the 5-year storm for the Richmond Transport area.

Shown on Figure 4-2 is an outfall priority discharge system for the entire west side. The Lake Merced, Mile Rock, and Sea Cliff Outfalls will be adequate for the 5-year storm. Overflows would never need to be discharged at Lincoln Way or Vicente Street. This finding is preliminary and subject to further study on system hydraulics.

Cost-Effectiveness Evaluation of Alternatives

The cost of achieving 10:1 dilution, over $36 million in capital cost, must be weighed against the minimal cost of using the existing outfalls in a prioritized system. Based on the above preliminary analysis, this latter option would result in overflows being discharged at the following locations and frequencies:

Lake Merced--eight times per year.
Mile Rock--eight times per year.
Sea Cliff--one time per year.
Vicente--zero times per year.
Bakers Beach--zero times per year.
Lincoln Way--zero times per year.

Table 4-5. Physical Features of Combined Sewer Overflow Outfalls for Westside Sewerage System

Number	Name	Size	Capacity, mgd, at head loss =		
			9.0 ft	11.0 ft	45.0 ft
1	Lake Merced	10 ft x 11 ft 3 in.	670	-	-
2	Vicente	2 at 5 ft diameter	-	435	-
3	Lincoln Way	6 ft x 6 ft 6 in. 2 at 6 ft x 6 ft	-	1,150	-
4	Mile Rock	9 ft x 11 in.	-	465	-
5	Sea Cliff PS No. 1	18 in. diameter	-	-	-
6	Sea Cliff	6 ft diameter	-	-	765
7	Sea Cliff PS No. 2	12 in. diameter	-	-	-
8	Bakers Beach	7 ft diameter	-	-	1,120

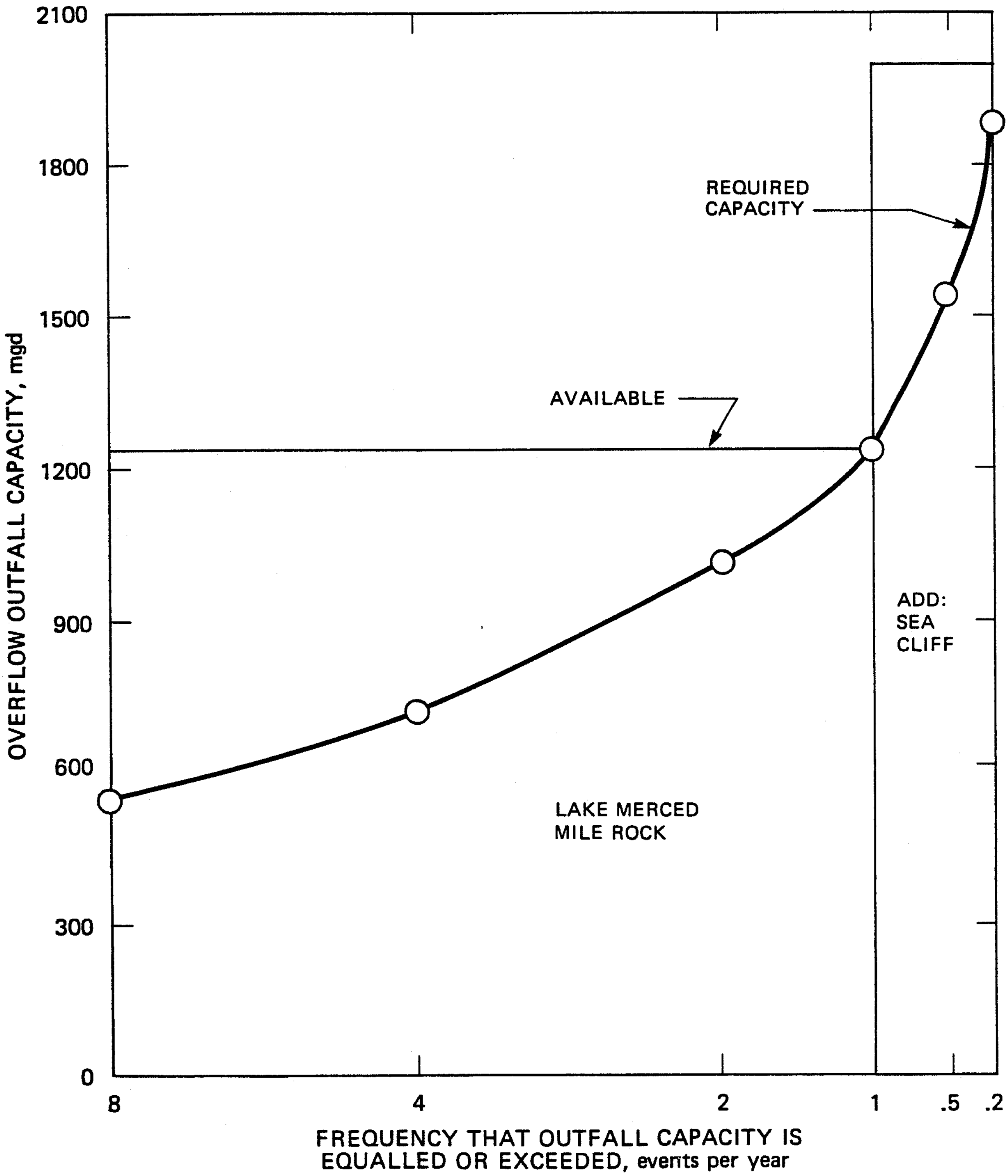


Figure 4-2 West Side Overflow Discharge Priority System

Based upon the coliform surveys and float studies, the impacted area from each overflow is approximately 2,000 feet of beach. Previous studies indicate that the impact on bacteriological water quality will persist for about 3 days following an overflow. Ignoring the fact that baffling the overflows to reduce discharge of floatables will improve overflow water quality, it can be assumed that beach areas will be impacted (i.e., water quality standards exceeded) for 24 days per year.

This limited impact must be weighed against the high cost of extending outfalls. Assuming no impact from an extended outfall, the improvement is about 24 days per year. Converting the capital cost of extended outfalls to an annual cost and dividing by the maximum number of days of improvement means that the cost will be $128,000 per usable beach day for the four beach areas listed above. Using information in Table 4-4, this cost per usable beach day equates to about $55 per beach user. This leads one to the conclusion that extended outfalls would not be cost-effective for the west side.

Recommendations

It is recommended that the 10:1 dilution requirement be eliminated and overflows be discharged according to the outfall location priority system described above. This recommendation is in accordance with the U.S. Environmental Protection Agency's opinion that extended outfalls are "very likely too expensive an option." This opinion was expressed in a letter to Mr. Larry Walker, State Water Resources Control Board, from Mr. Frank Covington dated August 17, 1979.

NORTH SHORE OVERFLOWS

In the following paragraphs, the most favorable locations for overflows are established for the North Shore followed by alternatives to achieve 10:1 dilution and/or prioritize discharges from existing outfalls. Recommendations are based upon cost-effectiveness.

Most Favorable Locations for Overflows

Specified beneficial uses (NPDES Permit No. CA0038610) for the North Shore between Baker Street and Jackson Street (Outfalls 9 through 17) are water-contact recreation, nonwater-contact recreation, wildlife habitats, marine habitat, ocean commercial and sport fishing, and fish migration. These uses, together with existing outfalls, are shown on Figure 4-1. The water-contact recreation is focused at the Aquatic Park on a year-round basis, where about 200 people swim each day. Less water-contact recreation occurs from Baker Street to Fort Point. There are three small boat harbors (St. Francis Yacht Club, Gas House Cover, and one near Pier 39) which have overflows discharged directly to them. The following order of most favorable overflow discharge locations have been established based upon protecting the principal beneficial uses:

Marina area

1. Baker Street (Outfall No. 9)
2. Pierce Street (Outfall No. 10)
3. Laguna Street (Outfall No. 11)

Embarcadero

1. Existing North Point Water Pollution Control Plant (NPWPCP) Outfalls
2. Jackson Street (Outfall No. 17)
3. Greenwich Street (Outfall No. 16)
4. Sansome Street (Outfall No. 15)
5. Beach Street (Outfall No. 13)

Based upon float survey data, discharge should occur first in the Embarcadero area before the marine area, if possible, to protect North Shore area beaches.

Alternatives

Shown in Table 4-6 are the sizes and estimated capacity for the existing outfalls. Available head loss is expected to be in the range of 1.0 to 2.0 feet depending upon overflow weir elevations in the new transport/storage structures and tidal conditions. For example, if overflow weirs are set at -5.0 and with a maximum water surface elevation in the transport/storage structures of -3.5, the available head loss is 1.5 feet for the outfalls.

The existing NPWPCP Outfall system is capable of about 160 mgd. Using the excess pumping capacity at the North Shore Pump Station provides an outfall capacity of about 100 mgd. Gravity flow through the outfalls from the North Shore transport/storage would probably provide significantly less capacity because less head would be available. These outfalls have diffusers which were designed to achieve 10:1 dilution.

The most preferable existing discharge would be to continue to use the NPWPCP Outfalls. Assuming the pumped outfall option, there would be costs for electric power from pumping for four overflows per year. Because the duration of pumping is only 13 hours per year, this cost is estimated to be only a few hundred dollars per year. There may be some capital cost involved in modifying the

Table 4-6. Physical Features of Combined Sewer Overflow Outfalls for North Shore Sewerage System

Number	Name	Size	Capacity, mgd, at head loss =		
			1.0 ft	1.5 ft	2.0 ft
9	Baker Street	9 ft diameter	155	190	220
10	Pierce	8 ft diameter	110	130	145
11	Laguna	6 ft diameter	115	140	155
13	Beach	6 ft x 7 ft	140	175	205
15	Sansome	2 at 5 ft 6 in. x 6 ft 6 in.	480	590	680
16	Greenwich	6 ft diameter	150	170	190
17	Jackson	8 ft x 9 ft 6 in.	290	355	420
-	North Point Water Pollution Control Plant	4 at 4 ft diameter	160	160	160

North Shore Pump Station to be able to use these outfalls. The cost has not been precisely estimated, but it is assumed to be on the order of several hundred thousand dollars.

The Jackson Street Outfall is the next best discharge location. It has a large capacity equaling 355 mgd at a head loss of 1.5 feet, which is assumed available. Initial dilution would probably be similar to that measured at Howard Street, viz. 16:1. No modifications would be required to use this outfall with the recently constructed North Shore system, except for weirs and controls.

An outfall in the Marina area will probably be needed. Of the three existing outfalls, the Baker Street Outfall is in the best location because it discharges 290 feet offshore. Dye studies done on the existing outfall indicated that initial dilution was at least 8:1. Float studies done on the Corps of Engineers Bay Model indicated that water movement in this area was sluggish and that eddies carried the water to the shoreline and along the beaches at Crissy Field. To get outside of these eddies would require that the discharge be relocated to at least 1,000 feet offshore. The Baker Street Outfall could be extended the additional 700 feet. The outfall was built in 1970 at a cost of $397,000. At today's prices, a 700-foot extension without a diffuser would cost $3.4 million. It is assumed that, because the outfall would discharge in 60 feet of water instead of the present 30 feet (MLLW) of water, 10:1 dilution would be achieved.

An extension to the Pierce Street Outfall would probably cost about the same as the new North Point Outfall contained in the report entitled "Bayside Overflows" prepared by CH2M HILL in June 1979. This new outfall would be a 8.75-foot-diameter pipe extending 1,760 feet into the bay with a 360-foot-long diffuser. This outfall would cost about $4.9 million (ENR 3597). Costs are about 6 percent higher today. It would appear cheaper to extend the Baker Street Outfall if greater dilution is desired, although an extension to Pierce Street outside the St. Francis Yacht Club would be a better location for discharge because it is farther from beach areas.

Shown on Figure 4-3 is the manner in which the existing outfalls could accommodate the projected overflows from new transport/storage facilities. The NPWPCP and Jackson Street Outfalls would be adequate for all four overflows occurring in the average year. Once every 2 years it would be necessary to use the Baker Street Outfall. This three-outfall system would be adequate for the 5-year storm as well. This conclusion is preliminary, subject to further study on the system hydraulics. It may be possible that an overflow would occur from the Marina area before the North Shore Pump Station could lower the water level in the transport/storage facilities by pumping through the NPWPCP outfalls.

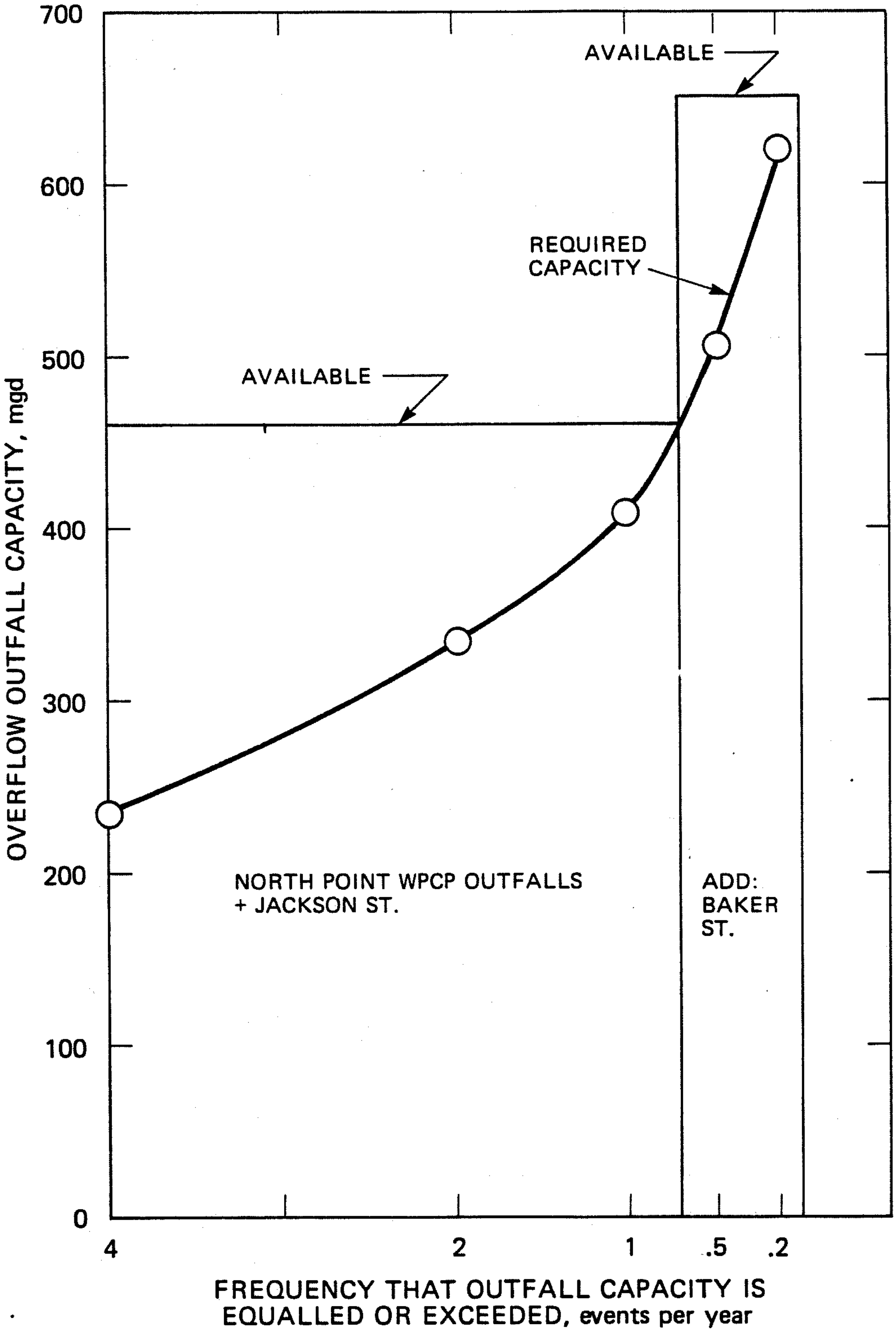


Figure 4-3 North Shore Overflow Discharge Priority System

Cost-Effectiveness Evaluation of Alternatives

The cost for using the existing NPWPCP Outfalls is much less than extending the Baker Street Outfall or building a new outfall. Based on an overall system hydraulic balance, it appears that the Baker Street Outfall or a new outfall would not be needed in an average year. Even if it were used four times per year for a few hours at a time causing an assumed 12 violation days per year, it would cost $27,000 per violation day to extend the outfall and eliminate violation days. It does not appear cost-effective to spend this much money to protect an occasional swimmer.

Recommendations

It is recommended that the overflows from the North Shore system be discharged through, in order of priority, the NPWPCP Outfalls, Jackson Street, and Baker Street Outfalls. Laguna Street and Beach Street Outfalls could be abandoned and the remainder be retained for use with very large storms. No new outfall construction is recommended.

CHANNEL OVERFLOWS

Priorities for discharge in the Channel area are described below followed by alternatives to eliminate discharge to Channel and achieve 10:1 dilution. An outfall discharge priority system is also developed. Recommendations are based on cost-effectiveness.

Most Favorable Locations for Overflows

Specified beneficial uses (NPDES Permit No. CA0038610) in the Channel area (Outfalls 18 through 28) are water-contact recreation, nonwater-contact recreation, wildlife habitats, marine habitat, ocean commercial and sport fishing, and fish migration. Locations of existing outfalls are shown on Figure 4-1. Outfalls 22 through 28 discharge to the Channel area, a dead-end slough. The principal beneficial use inside the Channel area is recreational boating. A small number of houseboats are located there. Overflows along the bay shoreline through Outfalls 18 through 22 should not interfere with proposed development of the South Embarcadero area into a fishing area with a promenade. Most of those outfalls are located under existing piers. Clearly, the discharge priority in the Channel area is along the bay shoreline, not directly into the Channel. The Howard Street Outfall was shown to produce more than 10:1 dilution. Consequently, it is not necessary to extend outfalls to the end of the piers to get 10:1 dilution.

Alternatives

Shown in Table 4-7 are the sizes and capacities of the 11 existing outfalls in the Channel area. They range in capacity,

Table 4-7 Physical Features of Combined Sewer Overflow Outfalls for Channel Sewerage System

Number	Name	Size	Capacity, mgd, at head loss =		
			1.0 ft	1.5 ft	2.0 ft
18	Howard	7 ft diameter	185	220	255
19	Brannan	6 ft 6 in. x 6 ft	170	205	240
20	Townsend	2 ft x 3 ft	18	22	25
21	Berry	15 in. diameter	1	1	2
22	3rd Street	2 ft 6 in. x 3 ft 9 in.	24	28	35
23	N. Side 4th Street	6 ft 6 in. diameter	110	130	140
24	5th Street	9 ft x 7 ft	200	245	280
25	N. Side 6th Street	6 ft diameter	105	125	145
26	7th and Division	4 at 8 ft 3 in. x 9 ft 6 in.	1,150	1,400	1,700
27	S. Side 6th Street	3 ft 6 in. x 5 ft 3 in.	45	55	65
28	S. Side 4th Street	2 ft 6 in. x 3 ft 9 in.	48	60	70

at a head loss of 1.5 feet, from 1 to 1,400 mgd. The Seventh and Division Streets Outfall, at the head end of Channel, has 44 percent of the total outfall capacity of the area. The bay shoreline Outfalls 18 through 21 have 448-mgd capacity. These outfalls would only be adequate for the eighth, ninth, and tenth largest storms of the year. Referring to Figure 4-4, there would be seven storms per year which would require a discharge to Channel. By prioritizing outfalls, these could be confined to between Fifth Street and the mouth of Channel in an average year. The large overflow structure at the head end of Channel would be reserved for storms with a return period greater than 1 year.

The report "Bayside Overflows" contains alternatives which would eliminate discharges to the Channel area and obtain either 5:1 or 10:1 dilution under worst case conditions in the bay. These alternatives consist of gravity and pumped outfall systems. The gravity system, for 10:1 dilution, would consist of two 18-foot-diameter pipes, each 7,460 feet in length, plus a 1,560-foot-long multiport diffuser. This outfall would cost $60 million (ENR 3597). Achieving only 5:1 dilution would save $7 million. Pumping would have a cheaper capital cost (about $13 million less for each system) but would require power for a 7,000-horsepower pump station.

In an effort to further improve this situation without expending large sums of money, an alternative was developed to provide additional outfall capacity along the bay shoreline. Each of the small outfalls at Berry and Townsend Streets could be replaced with, for example, a 7-foot-diameter outfall having approximately 200-mgd capacity, the same as Howard and Brannan Streets Outfalls. The cost of these outfalls, each less than 100 feet long, would be about $100,000. These four outfalls would provide 800-mgd capacity which would accommodate all but one storm per year. It is assumed that 10:1 dilution would be provided. This conclusion is preliminary and subject to more detailed hydraulic analysis. Because the Channel area is mostly low in elevation, the selection of outfall priority and capacity will have to be done carefully so as not to aggravate existing flooding problems.

Cost-Effectiveness Evaluation of Alternatives

The cost of constructing additional outfall capacity at Berry and Townsend Streets must be weighed against the advantages of reducing overflows to Channel to about once per year. Because the cost is reasonable, and the beneficial uses in the Channel area are increasing, these two additional shoreline discharges are judged to be cost-effective. The long outfall into the bay ($60 million project) is judged not to be cost-effective because it would cost the equivalent of $190,000 per reduced violation day.

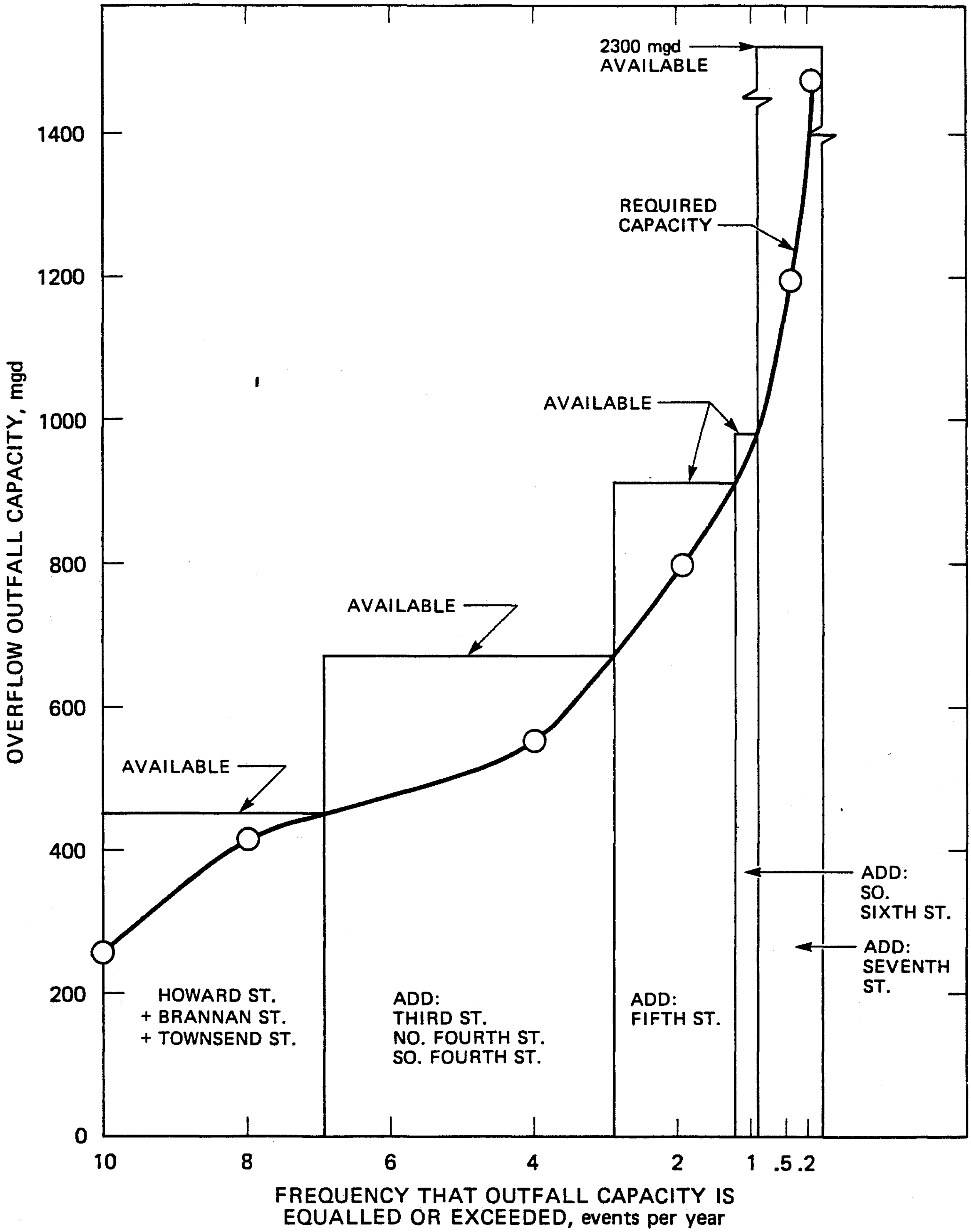


Figure 4-4 Channel Overflow Discharge Priority System

Recommendation

It is recommended that outfalls be prioritized and two new shoreline discharges be considered, one at Berry Street and one at Townsend Street. The priority would then be:

1. Howard Street (existing)
2. Brannan Street (existing)
3. Townsend Street (new)
4. Berry Street (new)
5. Third Street (existing)
6. North Side Fourth Street (existing)
7. South Side Fourth Street (existing)

The outfalls at Third and Fourth Streets would be needed only once per year. The outfall at Fifth Street would be needed once every 2 years and the Seventh and Division Streets Outfall would be needed once every 3 years.

MARIPOSA OVERFLOWS

Priorities for the Mariposa and 20th Street Outfalls are established below, and alternatives proposed to obtain 10:1 dilution, or otherwise improve the current situation, are developed. Recommendations are based on cost-effectiveness.

Most Favorable Locations for Overflows

Specified beneficial uses (NPDES Permit No. CA0038610) are the same as for North Shore and Channel. The principal beneficial use at Mariposa is nonwater-contact recreation. The overflow discharge is not well located, being in a marina next to the area of highest human activity. Practically, there is not a better place in the vicinity. A small park with a fishing pier is located to the north, and the area to the south is occupied by a privately owned shipyard. Consequently, an outfall extension or elimination of the outfall are the only better "locations." The 20th Street Outfall is small and located in a shipyard. No reasonable benefit would result from expending additional money to extend or relocate the discharge because access to the shoreline in this vicinity is restricted.

Alternatives

The first Mariposa alternative is to eliminate the discharge. This could most easily be done if a Channel-Islais low-level tunnel or other gravity connection to Islais Creek were constructed. The tunnel could easily handle the small overflow rates shown in Table 4-2. These volumes could also be accommodated by doubling the size of a new Mariposa pump station or oversizing additional storage. The additional costs for this system, designed for an additional 23-mgd capacity, would be about $5 million capital cost. This is a 70 percent increase in the amount of money to be spent in this area. An outfall extension of about 500 feet would cost about $1.7 million and might not provide 10:1 dilution under worst case conditions.

Cost-Effectiveness Evaluation of Alternatives

If the Channel-Islais low-level tunnel or gravity connection to Islais Creek is constructed, it would be cost-effective to accommodate potential overflows from Mariposa in this tunnel. The amount of overflow discharged into this area will be relatively small, probably less than 2 million gallons with each overflow. Oversizing Mariposa facilities, or extending the Mariposa Outfall, will not enhance beneficial uses significantly and would cost $5,400 to $16,000 per reduced violation day. Because of the lack of water-contact recreation in the area, the additional expense cannot be justified at this time.

Recommendations

It is recommended that overflows be accommodated in the Channel-Islais low-level tunnel, or a gravity interceptor, if constructed. Otherwise, oversizing Mariposa facilities to reduce overflows should be investigated in Bayside Facilities Planning, and the cost-effectiveness evaluated. No changes to the 20th Street Outfall is recommended.

ISLAIS CREEK OVERFLOWS

Priorities for the Islais Creek Outfalls are established and alternatives proposed to achieve 10:1 dilution and to remove the discharges from the dead-end slough. Recommendations are based upon cost-effectiveness.

Most Favorable Locations for Overflows

Specified beneficial uses (NPDES Permit No. CA0038610) for the Islais Creek area, Outfalls 31 through 35, are water-contact recreation, nonwater-contact recreation, wildlife habitats, marine habitat, ocean commercial and sport fishing, and fish migration. These uses, together with existing outfalls, are shown on Figure 4-1. Realistically, water-contact recreation and even nonwater-contact recreation are virtually nonexistent here. A small opportunity for recreation exists at the two mini-parks located at the Third Street bridge on Islais Creek. The parks are not used very much because of the lack of demand and lack of parking. Floats released just east of the Third Street bridge had less impact on Warm Water Cove and India Basin than floats released outside the mouth of the creek. Consequently, a discharge near the Third Street bridge is preferred over a discharge at the creek mouth and also over a discharge at the head end of Islais Creek.

Alternatives

Shown in Table 4-8 are the capacities of the existing outfalls discharging into Islais Creek (Outfalls 31 through 35). Referring to Table 4-2, the required capacities range from 308 mgd from the tenth largest storm of the year to 932 mgd for the largest storm of the year. Outfalls 31 and 35 discharge at Third Street but only have a combined capacity of 80 mgd at a head loss of 1.5 feet. Clearly, the majority of the capacity is with the Selby and Marin Street Outfalls. The existing 6-foot-diameter Southeast Water Pollution Control Plant (SEWPCP) effluent discharge line to Islais Creek has about a 70-mgd capacity. The new temporary outfall for the expanded SEWPCP has a capacity of about 180 mgd, where it enters Islais Creek near the Third Street bridge. These outfalls, if used to discharge overflows, would significantly add to the capacity of Outfalls 31 and 35. The four outfalls would handle the fourth largest storm of the year. Shown on Figure 4-5 is a prioritized outfall system using these outfalls plus Rankin Street first. These five outfalls could accommodate two of the ten overflows per year. The remainder of the overflows would be handled by Selby and Marin Street Outfalls.

The discharge locations could be improved if the northside Third Street Outfall were enlarged. If it were the same size as the temporary outfall for the SEWPCP, then six out of ten overflows in an average year could be handled by Outfalls 31, 35, the old and temporary SEWPCP Outfalls, and a new North Side Third Street Outfall. Rankin Street would probably not be needed with this system. The costs for this new outfall are estimated to be about $600,000. It could only be used if the transport/storage facility in this area were located nearby; the precise location has not yet been established.

The report "Bayside Overflows" contains alternatives which would remove discharges from Islais Creek and achieve 10:1 dilution or 5:1 dilution under worst case conditions. For a gravity outfall system achieving 10:1 dilution, two 17-foot-diameter pipes would be needed. They would extend 2,800 feet into the bay followed by a 1,560-foot-long diffuser. The costs for this alternative is $26 million (ENR 3597). To this would have to be added the land outfall costs, from the head end of Islais Creek to the bay shoreline, a distance of nearly 1 mile. Consequently, total costs would be considerably higher. The cheapest alternative

Table 4-8 Physical Features of Combined Sewer Overflow Outfalls for Southeast Sewerage System

Number	Name	Size	Capacity, mgd, at head loss =		
			1.0 ft	1.5 ft	2.0 ft
29	Mariposa	6 ft diameter	99	120	140
30	20th Street	24 in. diameter	-	-	-
31	North Side 3rd Street	3 ft 6 in. x 5 ft 3 in.	40	50	60
32	Marin	8 ft 10 in. diameter	200	240	275
33	Selby	3 at 7 ft x 10 ft	490	600	690
34	Rankin	5 ft diameter	65	80	90
35	South Side 3rd Street	2 ft 6 in. x 3 ft 9 in.	25	30	40
37	Evans	6 ft diameter	40	50	55
38	Hudson	30 in. diameter	10	15	20
39	Griffith North	21 in. diameter	5	10	10
40	Griffith South	5 ft 6 in. diameter	60	70	85
41	Yosemite	9 ft x 7 ft 3 in. and 11 ft 6 in. x 6 ft 6 in.	465	570	650
42	Fitch	6 ft 9 in. diameter	145	175	205
43	Sunnydale	6 ft 6 in. diameter	145	175	225
-	Old SEWPCP outfall to Islais Creek	6 ft diameter	-	70	-
-	New temporary SEWPCP outfall to Islais Creek	6 ft x 12 ft	-	180	-

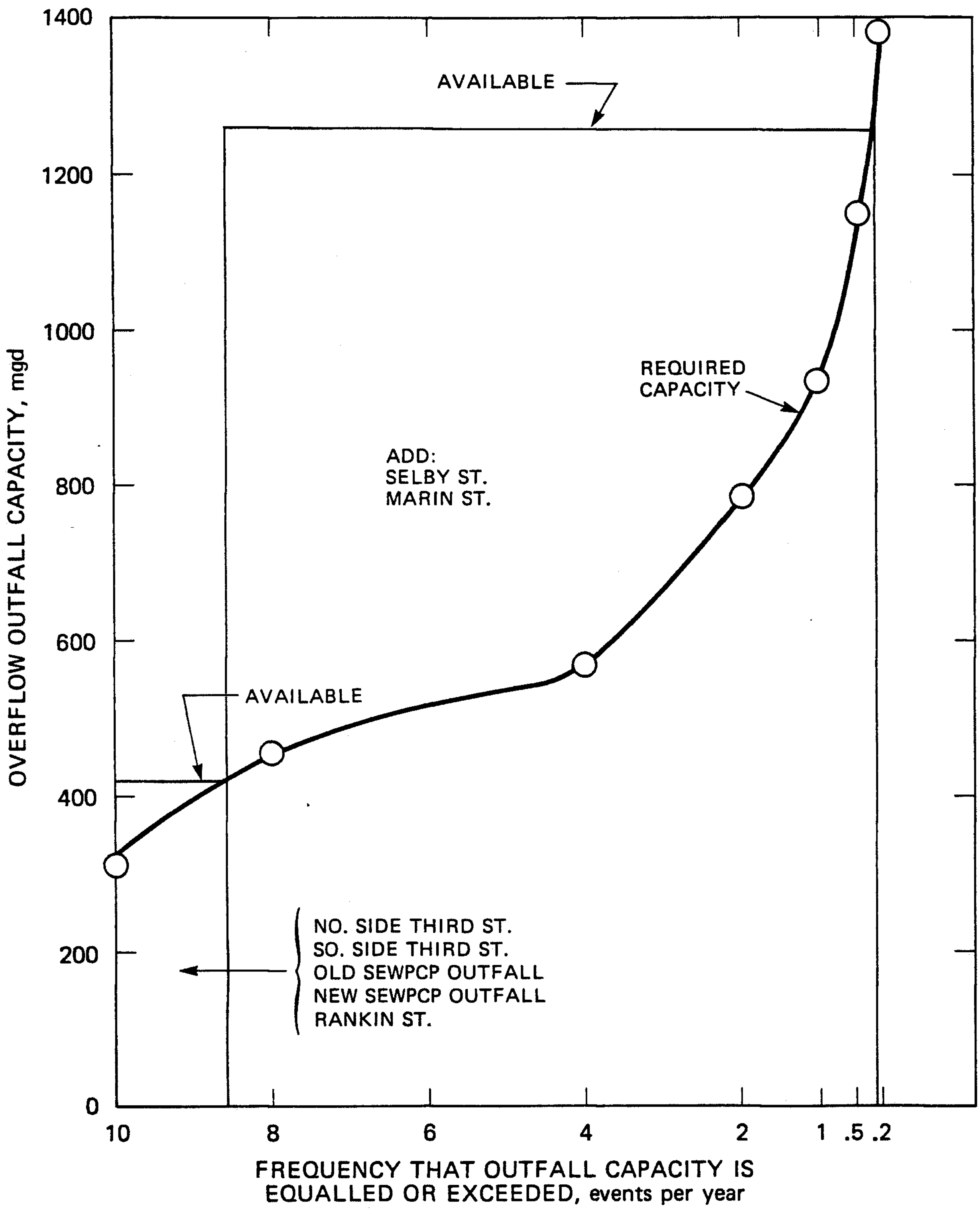


Figure 4-5 Islais Creek Overflow Discharge Priority System

evaluated for Islais Creek was a pumped outfall system achieving 5:1. This would have a capital cost of $16 million, but to this also must be added the land outfall cost and the annual power cost.

Cost-Effectiveness Evaluation of Alternatives

The costs of the large outfall system to divert overflows from Islais Creek cannot be justified at this time. Prioritizing the existing outfalls is clearly cost-effective. Additional outfall capacity on the north side of the Islais Creek near the Third Street bridge may be cost-effective. However, this would impact the two mini-parks at Third Street and may have more impact on Warm Water Cove than a discharge at the head end of Islais Creek. Further study is warranted on this alternative.

Recommendations

Existing outfalls, including the temporary SEWPCP Outfall, should be used in a prioritized system. Adding additional outfall capacity at the Third Street bridge should be investigated after the final configuration of the Islais Creek Transport/Storage facilities are determined in Bayside Facilities Planning Project.

EVANS-HUDSON OVERFLOWS

Priorities for the Evans-Hudson area are established for the specified one overflow per year. Alternatives are evaluated and recommendations made.

Most Favorable Locations for Overflows

Specified beneficial uses (NPDES Permit CA0038610) are the same as for the Islais Creek area, as shown on Figure 4-1. The Evans Street Outfall (No. 37) is farthest from the small boat harbor and principal recreation areas and should be used before Outfalls 38 and 39.

Alternatives

As shown in Table 4-2, virtually no overflow is expected in an average year. The capacity of any of the three existing outfalls is more than adequate. A slight oversizing of transport facilities could probably divert overflows to the Islais Creek area. No other alternatives have been evaluated or appear warranted.

Recommendations

It is recommended that the possibility of diverting overflows to Islais Creek, where they could be discharged near the Third

Street bridge, be investigated. Furthermore, the Evans Street Outfall should be used for any overflows which do occur in the Evans-Hudson area.

YOSEMITE-SUNNYDALE OVERFLOWS

Priorities for the Yosemite-Sunnydale area are established for the specified one overflow per year. Alternatives to achieve 10:1 dilution are evaluated and recommendations made based upon cost-effectiveness.

Most Favorable Locations for Overflows

Specified beneficial uses (NPDES Permit CA0038610) are the same as for Islais Creek with the important addition of shellfish harvesting. Nearly the entire Yosemite-Sunnydale area is being converted into a park, the Candlestick Point State Recreation Area. The Sunnydale overflow is located from the park and in an area of restricted access because of the freeway. The three Yosemite overflows will be located within the planned park. Plans for the park development in the Yosemite area are uncertain at this time due to a shortage of funds for the state parks. Consequently, on a preliminary basis, it appears that Sunnydale overflows should be discharged at Sunnydale and not transported to Yosemite. Within the existing Yosemite area, Fitch Street (Outfall No. 42) should probably be used first, Griffith Street (Outfall No. 40) second, and Yosemite Avenue (Outfall No. 41) last. This priority would attempt to divert overflows out of the South Basin Canal, a dead-end slough, and away from the proposed park nature area proposed for the north side of Yosemite Basin Canal. Priorities should be established only after park development plans are finalized.

Alternatives

Shown in Table 4-8 are the capacities of the existing outfalls. Comparing these with the required capacities in Table 4-2, it is clear that the existing Sunnydale overflow structure can easily handle the one overflow per year. Either Outfall 42, Fitch Street, or Outfall 40, Griffith South, has the capacity to handle the projected 1-year overflow rate from Yosemite. For storms with a larger return period, the Yosemite Avenue Outfall 41 would be needed.

The report "Bayside Overflows" evaluated outfalls to divert overflows from Yosemite Creek and to achieve either 5:1 dilution or 10:1 dilution under worst case conditions. For 10:1 dilution, an 11.25-foot-diameter outfall pipe, 6,060 feet long, plus a

960-foot-long diffuser, would be required. The cost was estimated to be $17.3 million. About $2 million could be saved by only achieving 5:1 dilution with a shorter diffuser.

If a reservoir is constructed at Yosemite near the mouth of South Basin Canal, an outfall could be constructed to the bay shoreline. For a 500-foot outfall to the shoreline with a capacity of 200 mgd, a 7.5-foot-diameter pipe would be required. The cost has been estimated to be $600,000.

Cost-Effectiveness Evaluation of Alternatives

The cost of a 7,000-foot-long outfall for Yosemite cannot be justified because it would only be used once per year and cost $550,000 per reduced violation day. Prioritizing outfalls is clearly cost-effective, but further study is warranted, especially since the proposed transport/storage system for this area has not been selected as yet, and the park development plans are tentative.

Recommendations

Outfalls should be prioritized. Sunnydale flows in excess of transport capacity should be discharged at Sunnydale. Tentatively, Yosemite overflows should be discharged from Fitch Street and/or Griffith South, thereby diverting overflows from this dead-end slough. If a reservoir is constructed north of Yosemite, a new shoreline discharge from the reservoir should be investigated. Final decisions on outfall priorities should be coordinated with park development.